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Session A2

Seismic Strengthening of Buildings: Case Studies
Renforcement parasismique des bâtiments: études de cas
Verstärkung bezüglich Erdbeben im Hochbau: Fallstudien

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Behaviour Assessment of Rehabilitated Buildings in Mexico City

Evaluation du comportement de bâtiments réparés à Mexico

Bewertung des Verhaltens wiederhergestellter Gebäude in Mexico City

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SUMMARY

After the earthquakes of 1985, over 1000 structures were rehabilitated in Mexico City. Due to the lack of technical guides available worldwide at that time, design and construction of retrofits were based on engineering judgement, intuition and experience. Different approaches were followed for solving similar problems; different degrees of safety were incorporated in analysis and design. A long-term research program is underway to assess the analysis, design, and construction considerations by studying the performance of rehabilitation schemes under future moderate and severe ground motions. Trends on the use of rehabilitation techniques for different types of structural systems have been identified.

RÉSUMÉ

A la suite des tremblements de terre de 1985, plus de mille immeubles ont été réparés à Mexico. Ne disposant à cette époque d'aucune règle technique dans le monde entier, la conception et la construction des réparations et le renforcement ont été réalisés sur la base de l'intuition et l'expérience des ingénieurs. Différentes approches ont été utilisées pour résoudre des cas similaires; différents facteurs de sécurité ont été incorporés dans la conception. Afin d'évaluer les hypothèses retenues dans le calcul, la conception et la réparation des bâtiments, un programme de recherche à long terme a été développé, en vue d'étudier le comportement des alternatives de réparation sous sollicitations de séismes, modérés et intenses. Certaines tendances apparaissent sur l'emploi des techniques de réparation pour différents type de systèmes structuraux.

ZUSAMMENFASSUNG

Seit dem Erdbeben von 1985 wurden mehr als 1000 Gebäude wiederhergestellt. Wegen der weltweiten Knappheit technischer Fachleute, basieren die Reparaturen auf Meinungen, Intuitionen und Erfahrungen von Ingenieuren. Unterschiedliche Ansätze wurden für ähnliche Probleme verwendet; unterschiedliche Sicherheitsfaktoren wurden gebraucht. Ein langfristiges Forschungsprogramm zur Bewertung der Berechnung, des Entwurfs und der Konstruktion von wiederhergestellten Gebäuden wird entwickelt. Tendenzen von Wiederherstellungstechniken wurden festgestellt.

1. BUILDING BEHAVIOR DURING THE 1985 MEXICO EARTHQUAKE

On September 19, 1985, an 8.1 magnitude earthquake struck the nation´s capital city. Although the focus was 400 km WSW from the city, the uniqueness of the ground motion intensity, frequency content, duration, and regularity was manifest in Mexico City. From the total building inventory of the city, only 1.4% collapsed or sustained serious damage. Most damaged and collapsed buildings were reinforced concrete (RC) frames and waffle slab structures, which proved the most vulnerable of all types of structures. An statistical summary of damage to buildings can be found elsewhere [1]. Cases of collapse or serious damage were mostly limited to buildings more than four stories tall. The most vulnerable proved to be those having 7 to 15 stories. The relationship between number of stories and vulnerability was explained in terms of the spectral shapes and the buildings' fundamental period of vibration.

For steel structures, primary causes of damage were local buckling or fracture of open-web members, local failure of box section columns, and inelastic buckling of slender cross bracing of old (pre-1957) structures [2]. In RC frame structures, the damage generally involved column failures due to high axial and flexural forces, shear failure in short captive columns, shear distress in beams due to large lateral movement or settlement of foundations, and joint distress due to inadequate confinement or poor layout of connected members. In RC slab structures, damage frequently was associated to shear distress near the column or at the edge of the "capital" over the column, punching shear, which probably initiated collapse of a number of floor slab structures, and flexural distress in columns.

Of the buildings that suffered collapse or severe damage, 42 percent were corner buildings. Most of these had masonry walls on two perpendicular sides and wide open facades to the street. In some cases, torsion was due to asymmetric layout of masonry filler walls. Weak first story failures were also due to an irregular distribution of masonry filler walls along the height, thus leaving the frames in the ground floor practically bare.

2. TYPICAL REHABILITATION SCHEMES USED IN MEXICO CITY

Over 1 000 buildings were rehabilitated after the September 1985 earthquake. Repair and strengthening of structures began almost immediately. Owners of damaged buildings were anxious to restore operations in their structures. A substantial number of buildings which suffered little or no damage were strengthened. In many cases strengthening was undertaken because similar buildings had collapsed or were heavily damaged. All repair and strengthening design had to meet the emergency building regulations (in effect until 1987), and since then, the current Mexico City Building Code [3]. Both demanded higher lateral forces and more stringent requirements to attain ductile behavior. Design requirements for rehabilitation of buildings were the same as those for new construction. Rehabilitated structures included schools and hospitals. The seismic safety of over 4 795 schools and 216 hospitals has been assessed. From the 1 687 schools affected by the earthquake, 1 658 structures with different levels of damage were repaired [4].

Techniques used for retrofitting were intended to both strengthen and stiffen the structures. In most cases, economic factors dictated the direction taking in rehabilitating the structure. However, least cost for construction was not always the most important consideration in selecting a rehabilitation technique. Many existing damaged and undamaged structures were rehabilitated at costs which would have exceeded demolition and reconstruction costs. This was done to preserve the amount of space that could be leased at premium rates because ordinances enacted since the structure was originally built would have required the inclusion of parking for occupants or a change in the use of the site.

The predominant rehabilitation techniques were column jacketing, addition of shear walls or diagonal bracing (designed to carry all or most of the lateral force), replacement of damaged elements, and removal of top floors. Relative merits and limitations of the techniques used have been discussed elsewhere [5]. At the time of the rehabilitation works, only qualitative design guidelines were available so that a great deal of engineering judgement and intuition were involved in the decisions regarding rehabilitation and severity of damage. Since then, several techniques have been assessed experimentally to provide a scientific basis for making such decisions [6,7].

3. RESEARCH PROGRAM

The National Center for Disaster Prevention (CENAPRED) in Mexico City has launched a long-term research program aimed at studying the behavior of rehabilitated buildings in the city, and at assessing the adequacy of the analysis, design, and construction considerations made for retrofitting. Monitoring the response of rehabilitated buildings in future ground motions will be carried out as part of this program. The project underway has been divided into four phases.

Phase I - Database of rehabilitated buildings. A database of some rehabilitated buildings in Mexico City was developed. To obtain relevant data from rehabilitated buildings a two-page questionnaire was prepared [8]. This questionnaire was sent to 15 design offices which have been involved in building rehabilitation in Mexico City. Collaboration of consulting firms was voluntary. Confidentiality of the information was warranted.

Phase II - Selection of typical buildings. From the buildings included in the database, a dozen structures are being selected based on a simple and symmetrical structural layout, typical rehabilitation scheme, availability of structural drawings, and importance of the structure. A complete record for each building will be prepared. Original structural drawings, damage information, and structural drawings of the rehabilitation will be included. At present two buildings, a school and a hospital have been selected.

Phase III - Assessment of selected buildings. Safety levels of selected buildings will be assessed by standard evaluation procedures. A first-level evaluation will be based on a visual inspection of the building for identifying characteristics that might be associated to substandard earthquake behavior. Items to be checked are the layout of structural system (both in plan and elevation), foundation characteristics, location, and damage. In a second-level assessment, a seismic safety index, based on cross-sectional areas of supporting (vertical) elements, will be compared to an intensity index, which reflects the seismic hazard of the zone where the building is located. In the third-level evaluation, the seismic capacity will be determined on the basis of current code provisions, and on state-of-the-art knowledge of behavior. Linear elastic and nonlinear analysis will be performed. An evaluation strategy for each selected structure will be developed so that it would be applied after future earthquakes. Ambient vibration tests will be performed to obtain the dynamic characteristics of buildings.

Phase IV - Post-seismic evaluation of selected buildings. At the occurrence of a moderate or severe ground motion, selected buildings will be inspected and evaluated. The performance of the rehabilitation schemes will be studied. Results from Phase III will be assessed in light of the responses observed. Two buildings will be instrumented with acceleration and displacement transducers.

4. PHASE I - DATABASE OF REHABILITATED BUILDINGS

Gathering of information has been a lengthy process. Since the cooperation of design firms has been voluntary, the time required to complete this phase has been dictated by the availability of consulting engineers. At present, 196 questionnaires have been received. Most structures (89%)

Activity	No. of Buildings	Percentage
Hospitals	16	9
Schools	138	81
Phone Stations	7	4
Mail Buildings	2	1
Offices	9	5

Table 1 Classification of Group A buildings

are located in the lake bed zone, characterized by soft clays, which is the area that showed the largest ground motion amplifications during the 1985 earthquake. The majority of the buildings are in zone most hardest hit by the earthquake.

Most buildings in the database are Group A structures (Table 1), which are defined by [3] as those which must be serviceable after an emergency and those in which large crowds may gather. In Table 2 building statistics based on type of vertical system are shown. Consistent with the damage observed in 1985, non-ductile RC frames (characterized by strong beam - weak column systems, wide tie spacing, flexible columns) and waffle slab structures are predominant. Schools are included in the non-ductile RC frame category.

Type of Vertical System	Frequency
1. Steel Frames	
a. Unbraced	13
b. Steel braces	1
c. RC walls	0
d. RC infills	0
e. Masonry infills	1
2. RC Frames	
a. Ductile detailing	1
b. Non-ductile details	140
c. RC walls	5
d. Masonry walls	13
e. Precast frame members	0
3. Masonry	
a. Reinforced	3
b. Confined	4
4. Waffle slab structures	
a. Without walls	34
b. With walls	4
5. Others	1

Table 2 Vertical system of sampled buildings

Regarding the floor system, 120 buildings have cast-in-place concrete slabs with beams. Most buildings have shallow foundations (164), which correspond to low-rise schools and clinics up to four stories high. As for deep foundations, 11 buildings are supported on bearing piles, while 21 are on friction piles.

The number of structures rehabilitated by the different types of rehabilitation techniques is shown in Table 3. It is clear that the techniques most widely used were those that strengthened and stiffened the structures. Jacketing of frame members, either with concrete or steel, and bracing of the building (with RC walls or steel rolled shapes) are predominant. A large number of structures in our sample were retrofitted with prestressing cable braces; this was the case of almost all schools and some low-rise hospitals.

Analysis of the information has shown that for RC frames with non-ductile detailing, jacketing of members was the technique most widely used (52 cases). In 11 of the 52 buildings, RC walls were added besides jacketing. Similar conclusions were reached for waffle slab structures.

Further assessment of the information indicates that in 85% of the cases, rehabilitation was performed to upgrade the structural characteristics to present code requirements although the structures were undamaged; this is closely related to the large number of group A structures in the

Type of Rehabilitation Technique	Frequency
1. Epoxy resin injection	17
2. Replacement of buckled reinforcement	6
3. Concrete jacketing	
a. Columns	15
b. Beams	0
c. Both	23
d. Joints	11
4. Steel jacketing	
a. Columns	13
b. Beams	0
c. Both	7
d. Joints	3
5. Addition of RC walls	33
6. Addition of RC infills	0
7. Addition of steel braces (rolled shapes)	25
8. Macroframes	6
9. Mortar cover reinforced with WWF mesh	11
10. Prestressing cables	128
11. Strengthening of floor diaphragm	0
12. Strengthening of wall-to-slab connection	9
13. Addition of piles	9
14. Jacketing of grade beams	14

Table 3 Statistics of the rehabilitation techniques used

sample (88%), which are required to fulfill the code [3]. Twenty-nine structures were retrofitted because they were damaged during the earthquake; it is important to point out that damaged structures were repaired soon after the event. Files for those buildings were not easily accessible in design firms; the majority of the structures sampled was rehabilitated after 1987.

The Mexico City Building Code accepts a reduction of the elastic design forces (elastic design response spectrum) based on inelastic behavior (ductility and energy dissipation) with a maximum response factor Q equal to 4. Q factors are 4, 3, 2, 1.5, and 1. As it is common, the largest the Q-factor, the more stringent are the reinforcing detailing requirements to attain ductile behavior. Detailing for Q=4 in RC structures is similar to that embodied in Chapter 21 of ACI- 318 code [9]. The vast majority of structures was designed for seismic effects using a response factor Q equal to 2. In this sample, Q factors equal to 3 and 4 were used in only nine structures.

5. CONCLUDING REMARKS

At this time, it is not possible to draw general conclusions. More information is expected from other design offices. Fruitful data will be obtained at the occurrence of future ground motions. However, it is evident that RC frames with non-ductile detailing and waffle slab structures are the types of vertical system with the largest number of retrofits. Jacketing and RC walls are the most common techniques in CENAPRED sample, along with bracing with prestressing cables.

The wide use of Q=2 for the design of rehabilitation schemes is consistent with the conservative approach followed by most structural engineers. Indeed, after the 1985 earthquakes, it was common that engineers relied on the original structure as capable for carrying vertical loads only, so that the lateral forces induced by the earthquake motions were to be resisted by the rehabilitation scheme added. In some instances, crude assumptions on the contribution of the existing structural system for resisting the lateral loads were made based on a great deal of engineering judgement.

Due to the relative low strength and low stiffness of the soil in the lake bed zone area in Mexico City, medium and high rise buildings required strengthening of the foundation; this was particularly the case when RC walls were added. Frequently, the foundation strengthening amounted for half of the total cost of the rehabilitation (including civil engineering work only), and evidently dictated the feasibility of the project. The existing structure imposed particular challenges for pile driving from the basement, and grade beam jacketing so that understanding the foundation behavior will be of paramount importance.

Although the progress of the project has been slow, results will certainly contribute to improve our knowledge on the behavior of rehabilitation schemes, and will aid in developing more reliable evaluation procedures and technical guidelines for building retrofitting.

6. ACKNOWLEDGEMENTS

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Seismic Repair and Upgrade of a Steel Braced Frame Building

Réparation et consolidation d'un immeuble à ossature en acier endommagée par un séisme

Reparatur und Ertüchtigung eines erdbebengeschädigten Stahlgeschossbaus

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SUMMARY

A modern four-story steel braced frame office building was structurally damaged in the 1994 Northridge, California, earthquake. The building, which was repaired and upgraded, provided an excellent opportunity to study the performance of a typical chevron braced frame system and to compare it analytically with other braced frame options. Linear and simplified nonlinear (static pushover) analyses were conducted on three different braced frame configurations, including the original system and the replacement system.

RÉSUMÉ

Un bâtiment moderne à ossature métallique de quatre étages, prévu pour des bureaux, a subi des dommages structuraux en 1994 lors du tremblement de terre de Northridge en Californie. La réparation et la consolidation de ce bâtiment ont permis de procéder à la comparaison analytique du comportement d'un système typique de cadre en treillis par rapport à celui de diverses autres ossatures du même genre. Il a été fait appel à des calculs statiques linéaires et non linéaires, en prenant en compte les effets de forces horizontales sur trois structures en portique raidies de manière différente, entre autres celle construite initialement et celle résultant de la solution adoptée pour la réparation.

ZUSAMMENFASSUNG

Ein modernes viergeschossiges Bürogebäude in Stahlbauweise wurde im Northridge-Erdbeben vom Januar 1994 in Kalifornien an tragenden Teilen beschädigt. Bei der Reparatur und Ertüchtigung ergab sich eine exzellente Gelegenheit, das Verhalten unterschiedlicher Ausfachungssystem auf analytischem Wege zu vergleichen. Dazu wurden lineare und statisch-nichtlineare Berechnungen, unter Horizontalkräften, an drei unterschiedlich ausgesteiften Rahmentragwerken, darunter das ursprüngliche und das ausgeführte Sanierungskonzept durchgeführt.

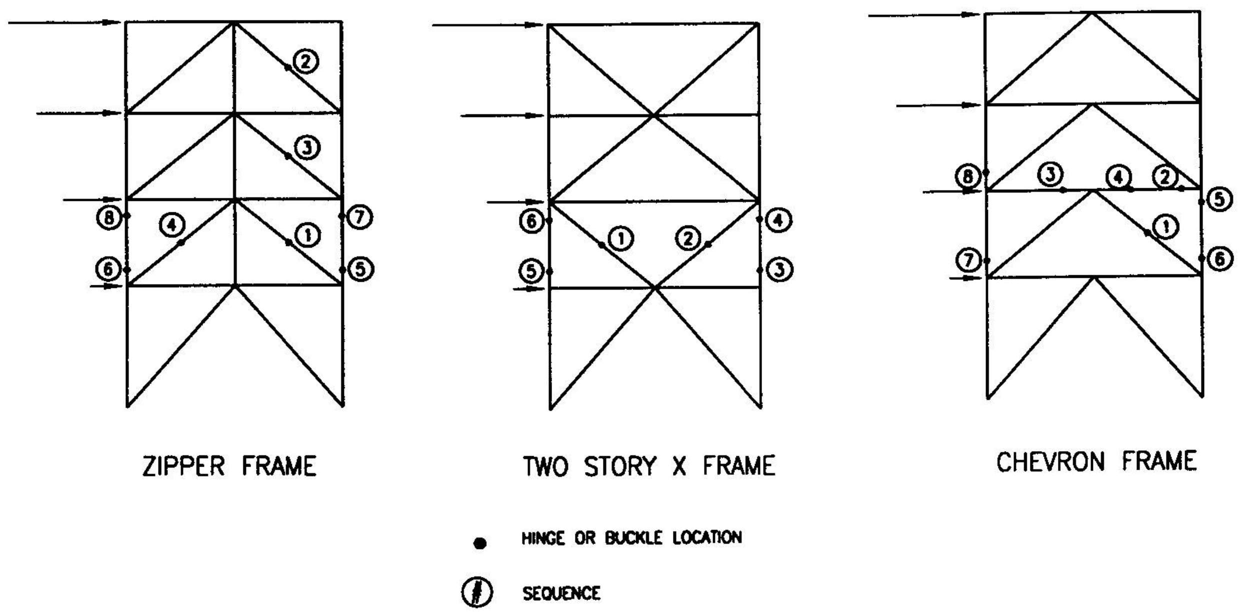
1. INTRODUCTION

The 1994 Northridge, California earthquake demonstrated the susceptibility of modern steel braced frame buildings to structural damage. An excellent example of ordinary, non-ductile, concentric braced frame (CBF) damage was provided by a four-story building in North Hollywood. This building, designed in accordance with recent seismic code provisions, suffered substantial damage in the Northridge earthquake and was repaired and strengthened in the following months. The authors were involved in the post-earthquake investigation, design of repair and strengthening, and construction administration. The building is particularly useful for study purposes since its structural system is typical of CBF's designed in accordance with U.S. Building Code requirements from the 1970s to the present.

The investigation of the subject building provided valuable insights into seismic performance from two standpoints: brace design and detailing, and braced frame configuration. Structural damage as well as design and detailing issues related to steel tube braces were previously discussed by Bonneville and Bartoletti [1]. This paper addresses braced frame configuration issues. During the repair and upgrade design, it was found analytically that post-buckling performance is affected substantially by the braced frame configuration. Therefore, the original analysis has been supplemented with additional studies, as described herein.

2. ANALYSIS APPROACH

Three braced frame configurations, based on the original building dimensions, have been studied. These include the original chevron configuration, a two-story X configuration, and a modified chevron configuration known as a zipper system [2]. For each configuration, a simplified nonlinear (static pushover) analysis [3] was conducted using the program SNAP-2DX [4]. This program was considered most appropriate for the analysis because it models nonlinear brace behavior more accurately than other software available. In the analysis, the structure is "pushed" to failure by incrementally increasing the static lateral loads until an instability forms. The static pushover analysis is a relatively simple way to determine progression of failure and relative deformations in a lateral system. In order to assess the affect of beam-column connections on the ductility of the braced frames, both fixed and pinned beam-column connections were analyzed for each frame configuration. The analyses were terminated when a complete mechanism formed in the frames and not necessarily at the point of collapse. Figures 1 and 2 show results for frames with fixed and pinned beam-column connections respectively. At the top of the figures, the three frame configurations are shown with the sequence of hinging and buckling of the members. Below, load versus deflection relationships are shown.



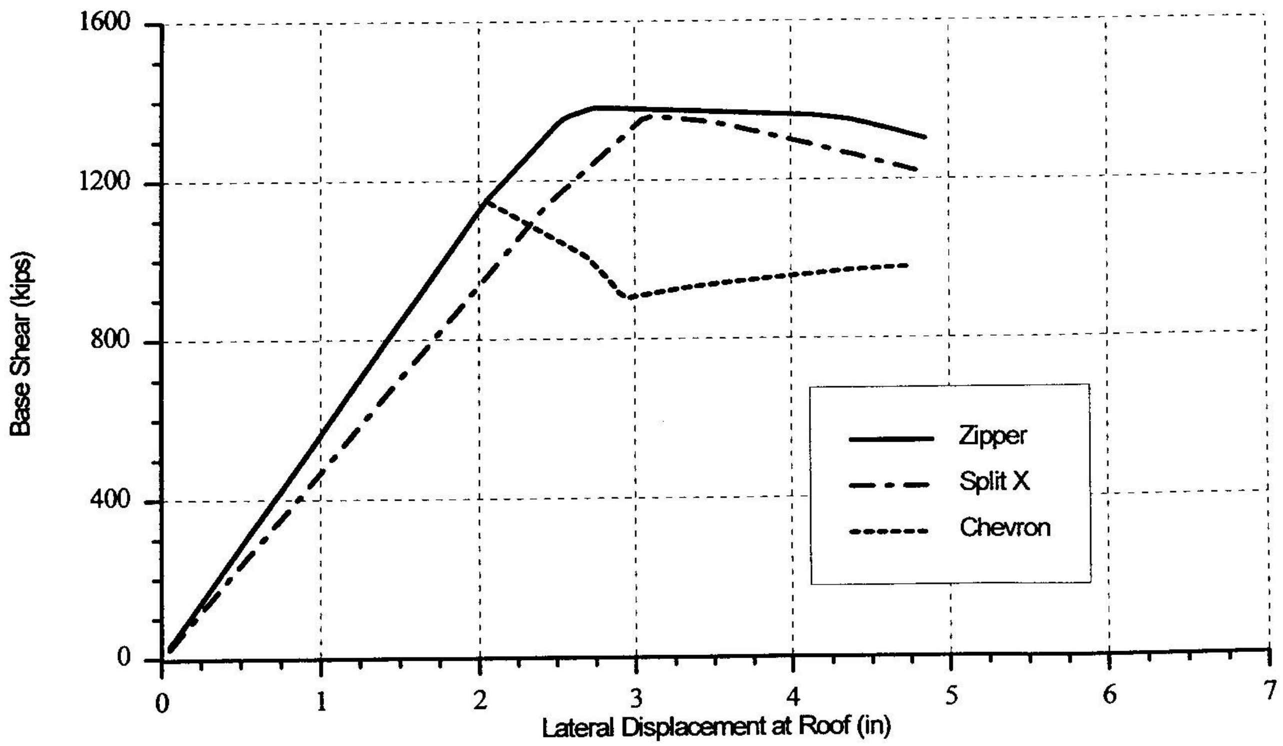


Figure 1 : Load vs Deflection - Fixed Girder to Column Connections

3. RESULTS OF ANALYSIS

3.1 Chevron Configuration

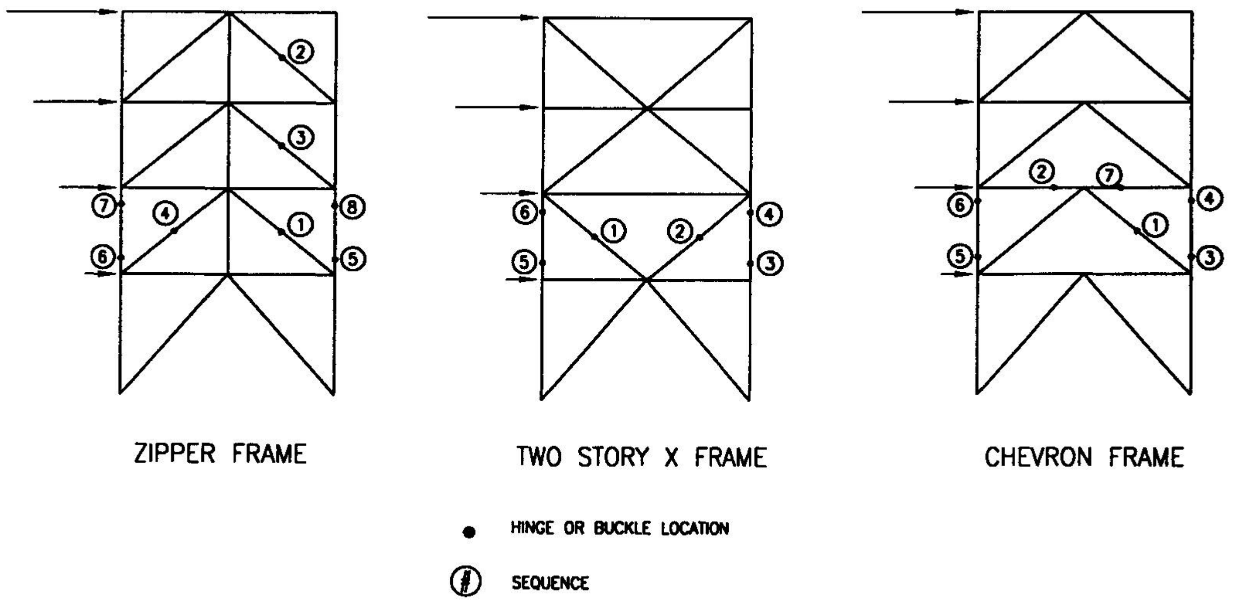
It should be noted that for the subject building, the ground level braces in all configurations were designed to be relatively stronger than the upper floors and, as a result, buckling occurs at the second level. Chevron brace configurations are generally thought to perform poorly in seismic events, thus the UBC [5] requires the brace members to be designed for 1.5 times the otherwise prescribed seismic forces. This requirement reflects the reduction in capacity that occurs for a bay of chevron bracing once the first brace buckles. For the frame studied, the capacity reduced by 20% for rigid beam-to-column connections and 30% for pinned connections. It was found that the post buckling strength and stiffness is highly dependent on the size of the beam intersected by the braces, which must resist the unbalanced vertical component of force between the tension and buckled compression brace. Improved performance can be obtained by using a beam strong enough to meet the UBC provision for special concentrically braced frames with a chevron configuration.

3.2 Two-Story X Configuration

The two-story X configuration was included in the analysis because it is a commonly used lateral system intended to alleviate the problems associated with the chevron system. When one brace buckles, lateral loads are transferred to other brace members rather than forcing the beam to resist the unbalanced vertical force in bending. In the analyses performed, the two-story X frame had a lower elastic stiffness than the other two configurations due to the distribution of overturning forces in the columns and, like the chevron, all yielding occurred in one story. In both the pinned and fixed configurations, a story mechanism formed through buckling and tension yield of the second floor braces followed by hinging of the second floor columns. After initial yield, the frame had additional capacity and reached a maximum capacity close to the maximum capacity of the zipper frame.

3.3 Zipper Configuration

The zipper braced frame is the system used for retrofit of the subject building. The design, based on research done by Khatib, Mahin, and Pister [2], is a modified chevron frame which utilizes vertical members at the beam midspans to redistribute lateral loads up the height of the frame. Figures 1 and 2 show that the zipper frame and chevron frame have identical elastic stiffness. However, while the capacity of the chevron frame drops off immediately after first yield, the capacity of the zipper increases until all three braces above the ground floor have buckled. Then, the second story tension brace yields and hinges form in the second floor columns to form a soft story. As seen with the other two configurations, the pinned condition has slightly less stiffness after first yield and a longer yield plateau as hinges form in the columns. In addition, the zipper



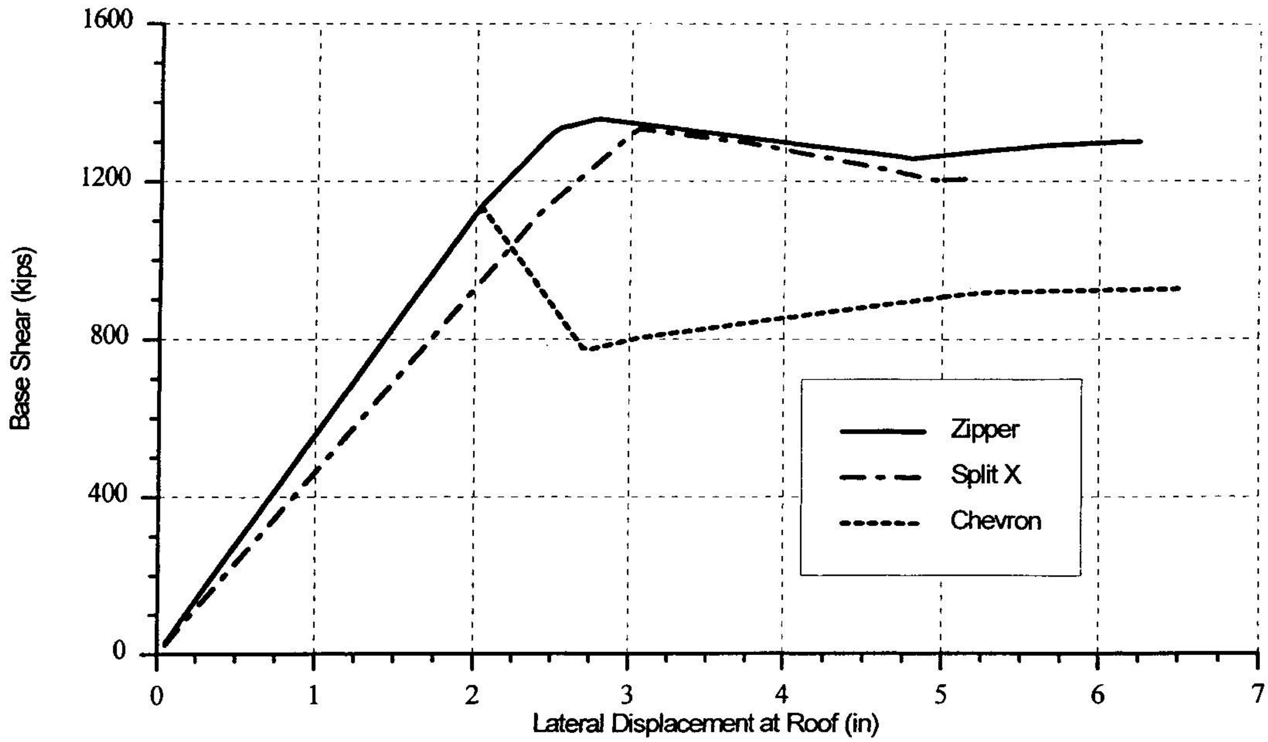


Figure 2 : Load vs Deflection - Pinned Girder to Column Connections

frame maintains greater lateral capacity than both the chevron or the two-story X configuration.

4. CONCLUSIONS

In comparing the different frames, it appears that the zipper and two-story X have similar maximum capacities and both have additional strength after the first yielding event occurs. However, the buckling of several braces up the height of the zipper acts to dissipate energy and distribute lateral deflections more effectively than the two-story X and chevron configurations. Ultimately, a soft story mechanism forms in each of the three frames through column hinging regardless of beam-column fixity. While the pinned frames were more flexible and formed mechanisms at higher lateral displacements, the frames with moment connections will absorb more energy under cyclic loading and have less inter-story drift.

It should be noted that the analyses show the load-deformation behavior of a single bay braced frame. The effects of load redistribution or stiffness of adjacent frame bays are not included in the study. In addition, the nonlinear behavior of the braced frames is sensitive to member sizes and material strengths. Much additional research is required to more comprehensively understand the nonlinear behavior of braced frame structures.

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Renovation of Seattle's Historic Paramount Theatre

Réhabilitation du théâtre Paramount à Seattle

Renovierung des Paramount Theater in Seattle

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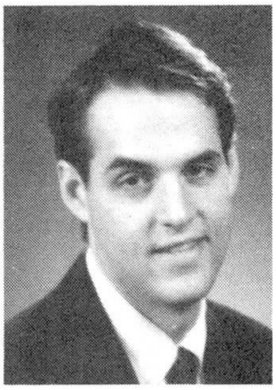

James Harriott was responsible for seismic design on the Paramount Theatre project. He has performed seismic analysis and design for numerous renovations, as well as for new construction. He received his BSCE from the Univ. of Washington and his MSCE from Univ. of California, Berkelev in 1990.

SUMMARY

The Paramount Theatre was built in the later 1920s in Seattle, Washington. Intricately designed by renowned local architect B. Martin Pritica, the 3000 seat theatre has hosted many millions of people over the years. Expectedly, the building has taken a fair amount of wear and tear. Although its beauty has aged gracefully, major renovations have become necessary to pass on the building's legacy to future generations. This paper presents the structural aspects of this renovation, including a major stage expansion, a backstage addition, and a seismic upgrade.

RÉSUMÉ

Le théâtre Paramount à Seattle, Washington, fut construit vers la fin des années vingt. Ce théâtre de 3000 places vit au cours des temps passer des millions de spectateurs. Aussi, les traces d'usure étaient-elles nettement visibles. Malgré un certain charme, son vieillissement a exigé dune profonde rénovation, afin que les générations futures puissent également en profiter. Les travaux de réhabilitation comportent l'agrandissement de la scène principale, l'adjonction d'une arrière-scène et le renforcement des structures contre les effets sismiques.

ZUSAMMENFASSUNG

Das Paramount Theater in Seattle, Washington, wurde in den späten Zwanzigerjahren erbaut. Das Theater hat mit 3000 Plätzen über Jahre hinweg viele Millionen Zuschauer gehabt. Entsprechend deutlich sind die Spuren der Abnutzung. Trotz seiner Schönheit hat die Alterung eine gründliche Renovation für die Uebergabe an zukünftige Generationen nötig werden lassen. Der Beitrag stellt die baulichen Arbeiten des Projekts vor, darunter eine deutliche Vergrösserung der Bühne, die Erweiterung der Hinterbühne und eine Verstärkung gegen Erdbeben.

1. INTRODUCTION

The Paramount Theatre (Figure 1) first opened in 1928 and was hailed by the national press to be the "largest and most beautiful theatre west of Chicago". Since that time, the facility has hosted numerous movies, vaudeville shows, concerts, Broadway performances, and presentations. Entertainment choices have changed dramatically over the past 65 years and the restoration is a response to those changes. The renovation includes a stage expansion that will accommodate the biggest Broadway shows, cleaning and restoration of public areas to bring them to their former glory, installation of state of the art sound and lighting systems, and the opening of new restaurants and clubs is planned. In addition, life safety improvements including seismic retrofit have been completed.

Fig. 1: Paramount Theatre

2. DESCRIPTION OF THE ORIGINAL STRUCTURE

The original building included a theatre and apartment. The theatre, where most of the current restoration is taking place, consists of a steel frame structure of riveted members built-up from plates and angles. The auditorium roof comprises trusses spaced at 6 m on center and free spanning the 36 m wide space. Two plate girders span the proscenium opening, one supporting the masonry fire wall above and the other the downstage portion of the gridiron and roof structure. The upstage portion of the gridiron and roof structure was originally supported on columns spaced at approximately 6 m on center. These columns are typical at the exterior theatre walls and are infilled with unreinforced brick masonry (URM). The interior theatre walls are steel frame infilled with unreinforced clay tile. The nine story apartment tower at the north end of the building is a reinforced concrete structure with pan joist floors, core walls, and exterior frames.

At the onset of the current restoration project, an extensive evaluation of the existing structure was undertaken. Material testing included compression strength of concrete cores, chemical analysis for weldability of steel, and in-place brick shear tests. Existing member sizes were spot checked in the field, and loading capacity of critical members was verified. Additionally, a comprehensive seismic evaluation of the existing structure was completed.

3. STAGE EXPANSION

The primary component of the theatre renovation was the stage expansion to accommodate modern, large performances. The stage width was increased from 23 m to 29 m and the depth was increased from 9 m to 15 m. In order to accomplish this expansion, major demolition of the existing structure was required. The demolition included complete removal of the stage back wall, removal of a two-story area at stage right, and partial removal of five stories of dressing rooms, stairs, and an elevator at stage left.

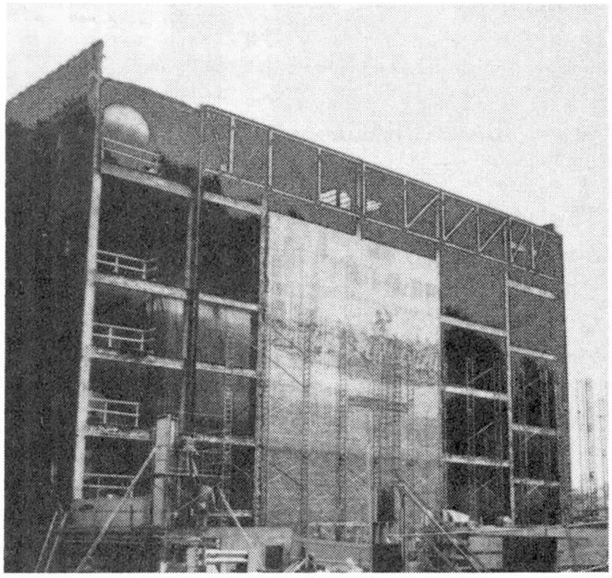

Fig. 2: New Truss

With the necessary removal of the back wall structure, the existing roof structure required new support. New support was provided by installation of a 3 m deep steel truss spanning 27 m over the new stage. The new truss can be seen in Figure 2. This photo was taken prior to demolition of the rear wall and existing columns.

Another important improvement during this renovation was the modification of the rigging arrangement. Originally, the stage rigging system was mounted to the gridiron 21 m above the stage and 3 m below the roof. This arrangement created a congested web of loft lines at the gridiron, making access to spot lines very difficult. In modern theatres, rigging consists of loft lines pulled vertically through sheaves which are attached to roof beams and hemp lines which thread through pulleys mounted on the gridiron. The loft lines support scenery and are spaced at 20 cm on center upstage and downstage and spaced at 3 m on center side to side. The loft lines are collected on the head block at stage right and then drop down to a counterbalance pit below stage level. The hemp lines support spot loads on the stage and pass through hemp head beams on each side of the gridiron. A plan view of the gridiron showing major elements is given in Figure 3.

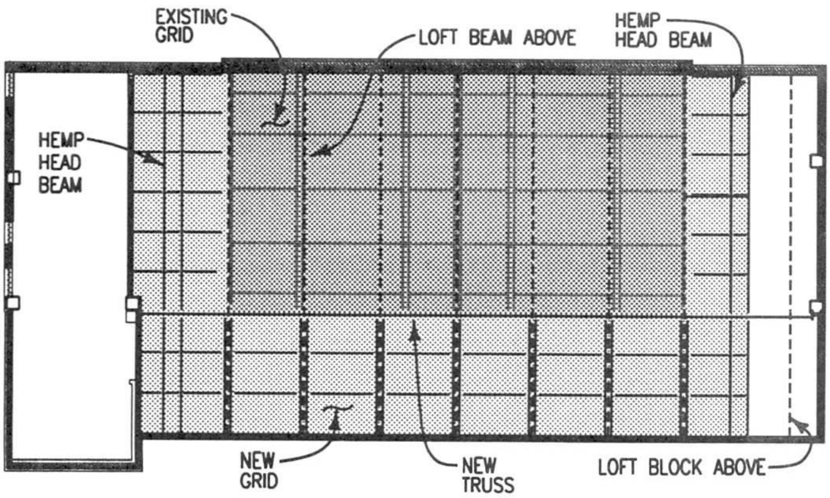

Fig. 3: Major Stage Rigging Elements

In general, the original roof and gridiron structure had adequate overall capacity for modern production loads, however, it was not configured for the new arrangement. Complete removal of the stage roof and gridiron structures was considered as an option, but cost and constructability issues dictated that the preferred approach was to modify the existing structure. This modification required careful consideration of sequence and load transfer during construction. A 9 step sequence was developed and was included on the structural drawings. The new elements to be installed included loft beams, head block, new gridiron hanger supports, gridiron grating and beams, hemp head beam, and two counterweight loading areas. The erection sequence allowed for installation of all the new elements without shoring the existing structure.

4. BACKSTAGE ADDITION

The next major component of the renovation was the expansion of the building in the area adjacent to the new stage. This expansion, approximately 9 m by 33 m in plan, included a new transformer vault and electrical room in the basement, a two-truck loading dock at street level, a large 6 m by 9 m freight elevator down to the stage level, a dressing room level, a passenger elevator and exit stair to allow access to 8 existing stage left levels, and a rooftop mechanical area.

A major excavation was required for the basement which was nearly 9 m below grade at some points. As is common for renovation work located in urban locations, this excavation required shoring around the perimeter. The shoring work was particularly challenging because it occurred in a cramped space adjacent to two city streets and a freeway exit ramp. Special tieback regrouting methods were used because of poor soil conditions and obstructions which limited anchor lengths.

The foundation of the new structure consisted of concrete augercast piles in combination with the steel soldier piles, which were attached to the concrete walls for transfer of permanent gravity loads. The loading dock is a cast-in-place concrete structure designed for highway truck loads. The dressing rooms and roof are conventional steel frame floor systems supported on steel tube columns at the exterior and concrete masonry unit (CMU) walls at the backstage wall. This tall slender 19 cm CMU wall also supports a portion of the expanded stage roof. A photograph of the building showing the backstage addition is given in Figure 4.

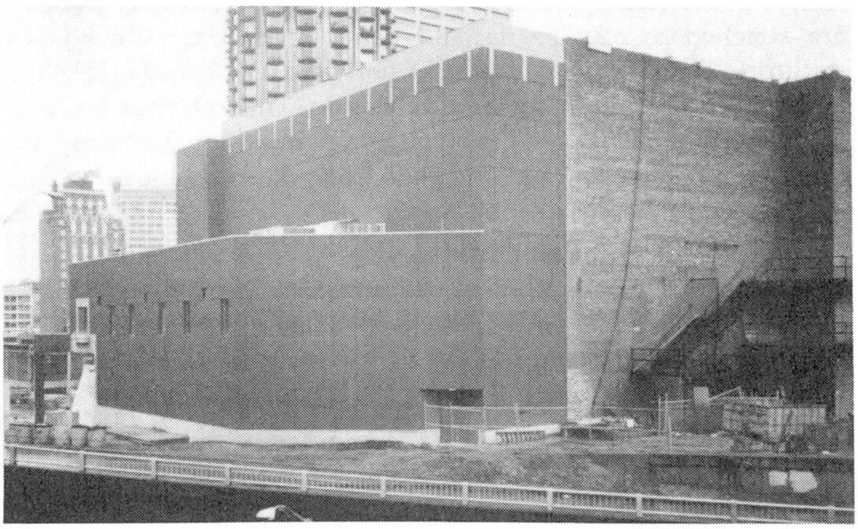

Fig. 4: Backstage Addition

5. SEISMIC UPGRADE

The seismic upgrade of the Paramount Theatre consisted of two main phases. The first step was to make a careful evaluation of the existing structure to identify local seismic hazards such as unbraced parapets and global seismic force resisting system deficiencies such as overstressed masonry walls. The second step was to craft a new lateral system which could supplement the strength of the walls in the existing building, as well as take advantage of the inherent large amounts of new masonry walls in the new addition.

The theatre was evaluated using the methodology of the "NEHRP Handbook for the Seismic Evaluation of Existing Structures," published by the Federal Emergency Management Agency. This handbook is commonly known as FEMA-178 and assists the designer to identify various local hazards in addition to the global lateral force resisting system deficiencies. The force level associated with the FEMA-178 analysis is somewhat lower than that of the UBC and is meant to provide a minimum life safety capacity for the lateral system. From the FEMA-178 evaluation process the following hazards were identified:

- The roof parapets were unbraced unreinforced masonry.
- The span of some URM walls between supporting steel girts or columns was excessive.
- The projection room above the auditorium was an unbraced hanging structure.
- Hollow clay tile walls existed at several locations above the auditorium.
- A weak story condition existed at the theatre lobby / apartment section of the building.
- There was excessive shear demand at the URM proscenium wall.
- There was excessive shear demand at the URM east and west exterior walls.

Providing for the mitigation of the local seismic hazards noted above was fairly straightforward once the hazards were identified. The URM roof parapets were braced to the roof using a traditional framework of angles. In this case, however, the URM was constructed of an open latticework of cast stone. To maintain the historic appearance of the lattice the cast stone was attached to the angle braces via epoxy anchors and chain link fence. Chain link fence was also used to mitigate the hazard of clay tile partitions above the auditorium ceiling. Chain link fence provided a means to control the debris from the expected clay tile failure in a cramped space next to an historic plaster ceiling.

The backstage addition was designed to resist all of the seismic loads resulting from the weight of the new building according to the 1991 Uniform Building Code (UBC). The lateral system in the new addition consists of reinforced masonry and concrete shearwalls. This structure is a stable and complete lateral force resisting system, without incorporating the existing elements. However, the walls in the new addition were also designed to assist the existing masonry walls and mitigate global deficiencies.

For seismic loads in the longitudinal (north/south) direction, the reinforced concrete shear walls in the new addition supplement the east and west URM walls in the existing building. The new concrete walls were tied directly to the balcony structure to transfer the large inertial forces collected in the balcony. These walls were designed for the greater of (1) the shear from the new addition based on the 1991 UBC, or (2) 25% of the shear from the entire building based on FEMA-178. To that extent the concrete walls serve as a back-up to the existing URM walls. The FEMA-178 R factor is 1.5 for unreinforced masonry shearwalls and 4.5 for masonry shearwallls. Due to the back-up reinforced shearwalls and because of the significant tensile capacity of the steel frame structure within the walls, the design of the east and west walls was based on an intermediate R factor of 2.5.

For seismic loads in the transverse (east/west) direction, new reinforced masonry and concrete shearwalls were designed to resist the entire seismic base shear. Since the existing backstage (south) wall was removed to accommodate the stage expansion, the new backstage wall of reinforced masonry was designed to resist the seismic forces from the southern portion of the combined building. A new shotcrete wall was placed against the demising wall between the theatre and apartment tower and was designed to resist seismic forces from the northern portion of the theater as well as the apartment tower. This wall eliminated the weak story condition in that portion of the building. In addition to the two main walls, the proscenium wall was strengthened. This strengthening was designed only to provide some measure of ductility at the proscenium opening. This wall was not designed to carry any of the building seismic forces apart from its own weight.

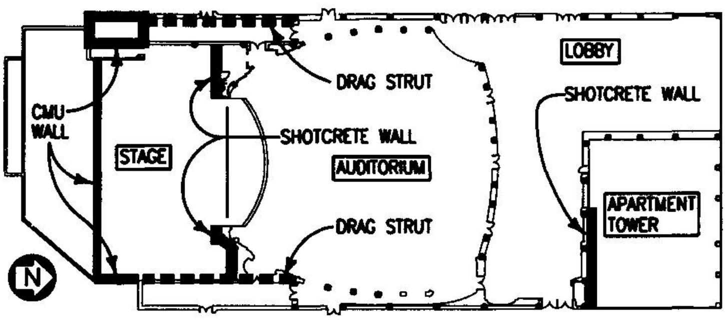

Fig. 5: New Seismic Elements

In summary, the seismic upgrade was designed to achieve two goals. First, the local seismic hazards such as hollow clay tile partitions and walls and unbraced parapets were mitigated. Second, the more serious deficiencies in the global seismic force resisting system were mitigated in a manner which took advantage of the configuration of the new addition as well as the inherent strength in the existing walls. The new concrete shearwalls were located at the walls demising the different functions of the building in order to minimize the impact to the architectural program. A key plan showing major lateral force resisting elements is given in Figure 5.

6. SUMMARY

In conclusion, the renovation of the Paramount Theatre was a challenging endeavor for the owners, contractors, architects, and engineers alike. With the motivation of a deadline for the opening of a major Broadway show, the construction was successfully completed at budget in an aggressive six month schedule. The load-in and rehearsals for Miss Saigon began in late January with opening night scheduled for March 17, 1995. In an era where many of the old theatres in the United States have been demolished, the perseverance of the owners and the skill and ingenuity of the design and construction team has revitalized a Seattle landmark for many years to come.

Seismic Retrofit of Allstate Building

Consolidation parasismique de l'immeuble Allstate

Seismische Ertüchtigung des Allstate-Gebäudes

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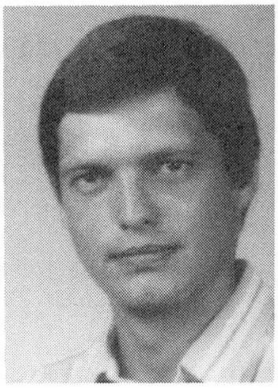

Uwe E. Dorka, born in 1956, has a Masters from the Univ. of Washington, Seattle, and a doctorate from the Ruhr-Univ., Bochum. He researched structural control systems for earthquake protection in USA, Japan, Austria. He now heads the Structures Lab. at the Univ. of Kaiserslautern.

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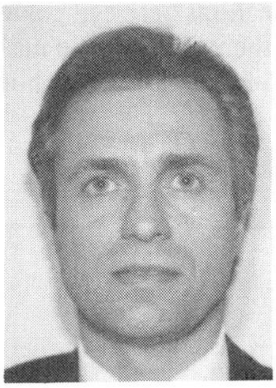

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SUMMARY

The seismic retrofit of the Allstate Building in Seattle is presented. The retrofit used a new structural system concept called the hysteretic device system. This system concept can limit story shear and drift to levels only known from very ductile or very stiff systems, respectively, thus combining the advantages of both conventional approaches. In the case of the Allstate Building, this rendered upgrading of the existing brittle reinforced concrete frames unnecessary, resulting in substantial cost-savings and improved performance when compared to conventional systems.

RÉSUMÉ

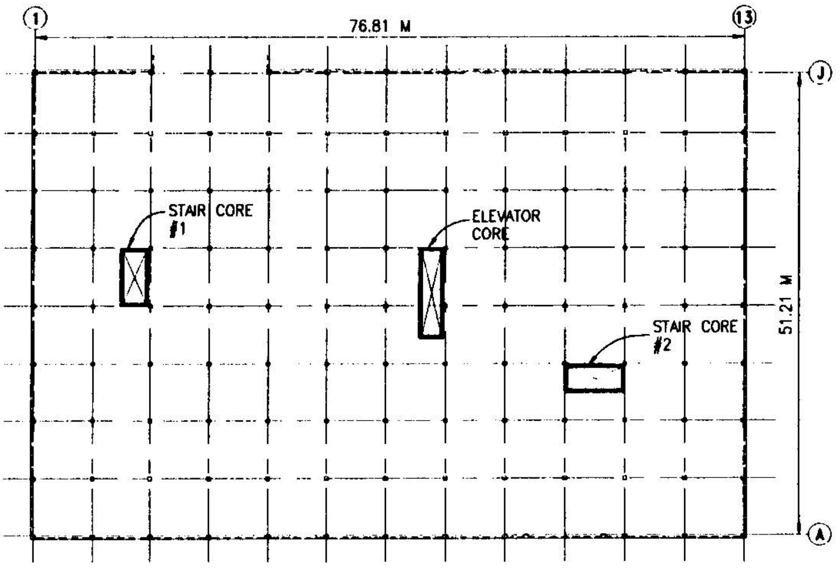
Cet article présente une nouvelle conception appelée "hysteretic device system", utilisée avec succès dans la consolidation parasismique de l'immeuble Allstate, à Seattle. Ce système permet de limiter les efforts et les déformations au cisaillement par étage successif, ceci à des ordres de grandeur atteints uniquement par des systèmes très ductiles ou très rigides. Dans le cas présent, il a été possible de renoncer à consolider les cadres en béton armé existants et relativement fragiles, entraînant une économie sensible des coûts de consolidation et une amélioration du comportement.

ZUSAMMENFASSUNG

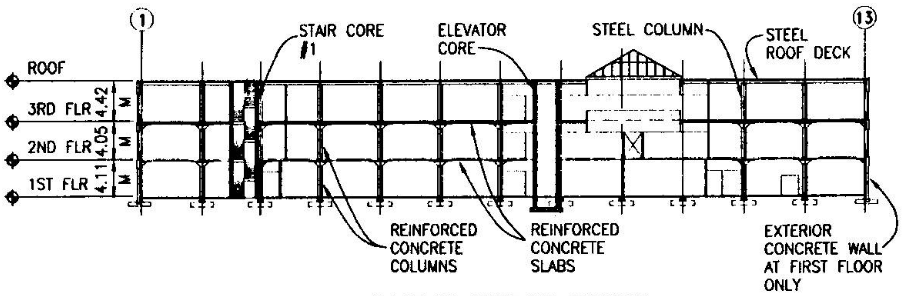
Es wird die seismische Ertüchtigung des Allstate-Gebäudes in Seattle gezeigt, bei der ein neues Tragwerkskonzept genannt "hysteretic device system" erfolgreich angewendet wurde. Dieses Konzept ermöglicht die Begrenzung der geschossweisen Schubkräfte und -deformationen auf Grössenordnungen, die nur von sehr duktilen Systemen einerseits bzw. sehr steifen Systemen andererseits erreicht werden. Es konnte dadurch die Ertüchtigung der vorhandenen spröden Stahlbetonrahmen entfallen, was erhebliche Kosteneinsparungen zur Folge hatte und zu einem verbesserten Verhalten führte.

1. INTRODUCTION

Built in 1959/60, the Allstate Building in Seattle is a three-story administration building that went through remodeling in 1991/92. FIG. 1 shows the plan and section of the building showing two new stairs and a new elevator shaft as well as a new glass-roofed atrium for the 1st and 2nd floor. The previous (and still existing) horizontal load resisting structure varies severely from floor to floor: The first floor has a stiff exterior concrete wall on the two short sides and one long side of the building which is needed as earth retaining wall. The second floor has a rc-frame formed by the columns and the slab and the 3rd floor has a light steel frame at the building's perimeter formed by steel columns and light steel trusses. The structural assessment of this system revealed several possible brittle failure modes. Built according to the relevant codes of that time, the rc-frame lacks ties in the columns and has weak connections between columns and slabs (FIG. 2) regardless of the solid concrete capitals. Although there is very little mass in the roof, the light steel perimeter frame is too soft and prone to connection failure as well as buckling of diagonals.



ALLSTATE BUILDING PLAN



ALLSTATE BUILDING SECTION

Fig. 1 Allstate Building floor plan and section with new rc-cores containing seismic links with shear panel dampers just below the 2nd floor ceiling.

Except for the stiff first floor system which will not exhibit any considerable story drift during an earthquake, the 2nd and 3rd story horizontal systems lacked the ductility required by modern codes. Therefore, a seismic retrofit became necessary.

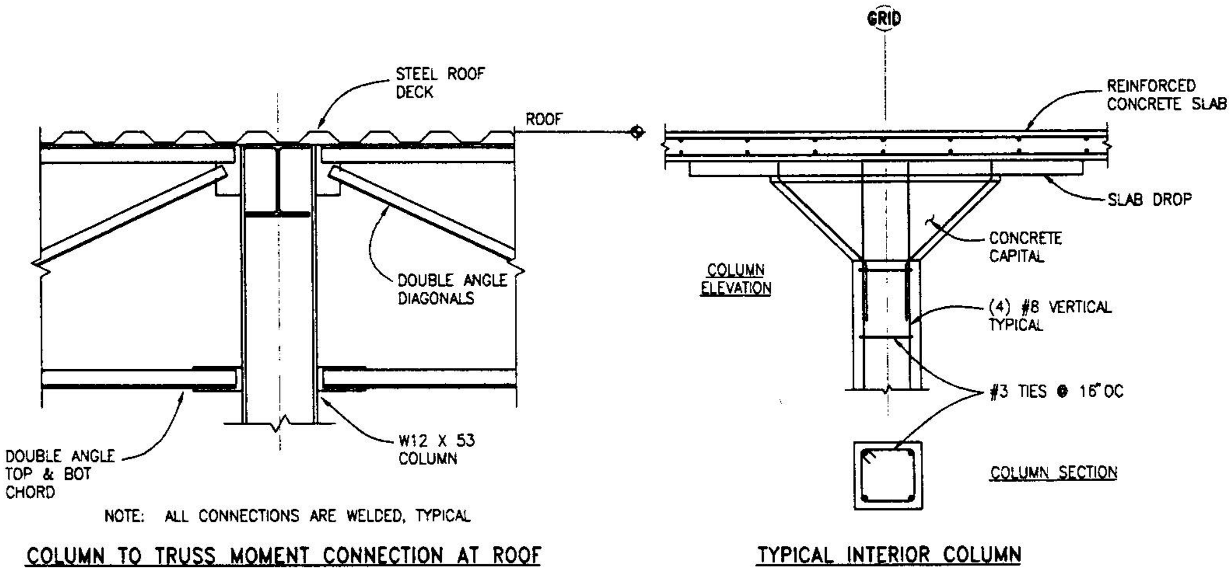


Fig.2 Details of existing framing system.

2. A NEW RETROFITTING SCHEME: THE HYSTERETIC DEVICE SYSTEM

Conventional retrofitting called for an upgrade of the rc-columns (confinement with an external layer of reinforcement) and joints in the existing frames and the implementation of new shear walls or truss systems. The new stair and elevator shafts provided only limited space for new conventional stiffening systems making the detailing of such systems very difficult and costly. In this situation, a Hysteretic Device - or Hyde-system (1) brought the solution.

Hyde-systems consist of a stiff primary horizontal system (PH-system), horizontal seismic links within the PH-system where hysteretic devices (Hydes) such as yielding or friction devices are placed and a soft secondary horizontal system (SH-system), where the masses are located.

Because of the stiffness of the PH-system, horizontal displacements of this system are concentrated in the seismic links where they activate the Hydes. These limit the maximum forces possible in the PH-system to their respective yield or friction force and dissipate most of the input energy. Both characteristics are very important for the structure: The physical force limit allows a static design approach and the use of very efficient structures for the PH-system and the large dissipation in combination with a large stiffness reduces story drifts. Studies (2) have shown that Hyde-systems are able to reduce story shears to very ductile system levels and story drifts to very stiff system levels, thus combining the benefits of both conventional approaches without many of their drawbacks. In addition, the seismic links can be designed such that, the Hydes are accessible and easily replaceable without major repair on the main structure. The performance of the structure is enhanced further, if well engineered yielding or friction devices are used (1), many of which were not available at the time of the Allstate Building's retrofit.

The SH-system has the important task to stabilize the P-Δ effect. Since during a major event, the Hydes are active most of the time, the PH-system provides little (if any) stabilization because its overall stiffness is near zero most of the time. Without an adequate SH-system, unpredictable and localized failure modes are possible (2).

In retrofitting, the existing system is often sufficient to act as SH-system. This is the case in the Allstate Building. Here, the new stair and elevator rc-cores provide the PH-system. The seismic links in each core are placed just below the 2nd floor ceiling (FIG. 3).

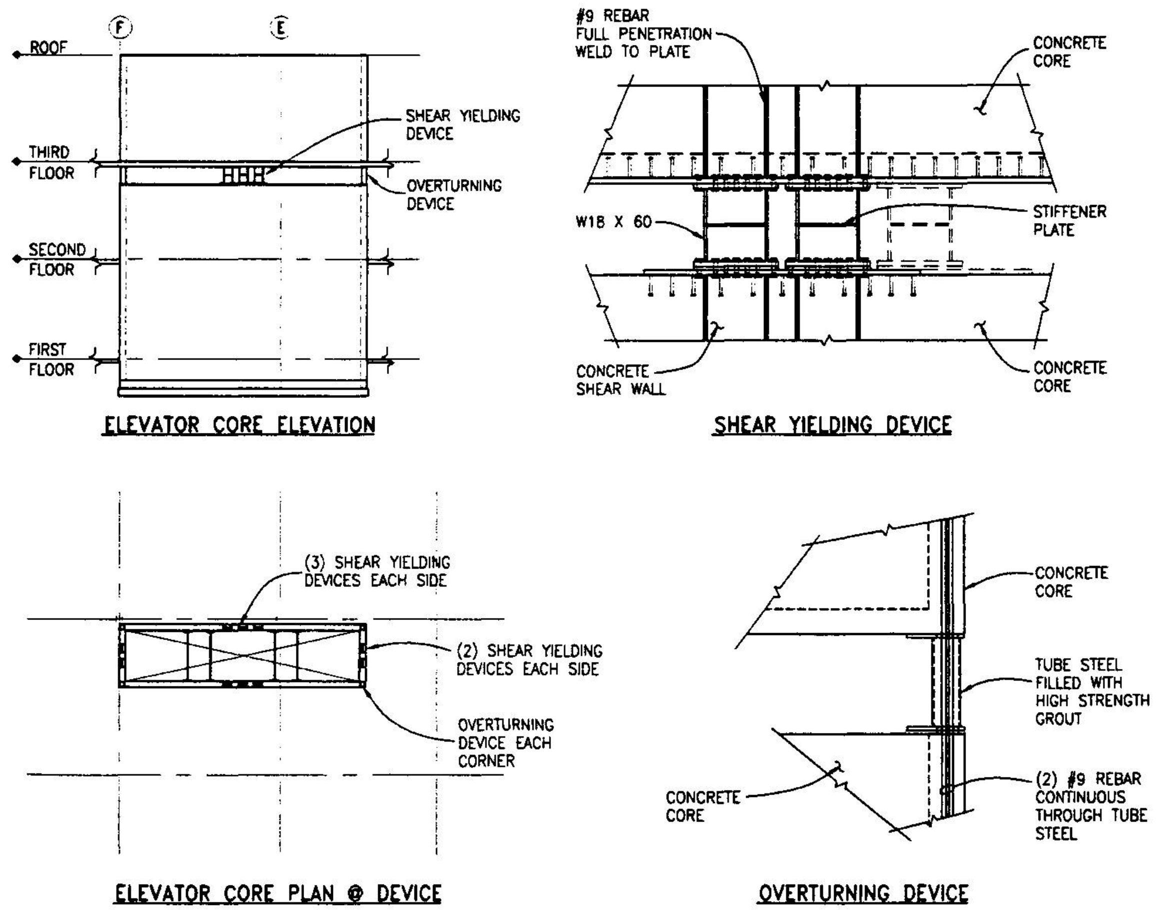


Fig.3 Elevator core as part of the primary horizontal system (PH-system) with seismic link below the 2nd floor ceiling. Shear panel dampers as hysteretic devices and grouted steel tubes in corners as overturning devices.

Only one link is necessary because the small roof mass can be coupled directly to the 3rd floor mass by the rc-cores and no story drift is expected in the 1st floor because of the stiff rc-perimeter walls there. As Hydes, shear panels cut from W18x60 were used. To prevent overturning, concrete filled steel tubes were placed in the corners of each link. These "overturning devices" yield during cyclic deformation of the seismic link without loosing their vertical load carrying capacity (tension and compression). Thus, the maximum horizontal yield force in each link is provided by the shear panels together with the overturning devices.

Because of its importance for the building's performance, the seismic links must be well detailed. Here, aspects like ease of inspection and replacement of devices are important. The links should be easily accessible and connections between devices and structure designed with additional capacity which depends on the possible variations in the devices' limit forces. Therefore, devices with well-known limit forces of small variation are preferable in Hyde-systems allowing for a more economical design not only of the connections but also of the complete PH-system. In the Allstate Building, the shear panels are bolted to the structure for easy replacement after a major event.

3. VERIFICATION

To verify the new PH-system, the UBC (3) provisions for excentrically braced frames (EBFs) were used because of the similarity of both systems: EBFs are also stiff-ductile systems with shear panels as hysteretic devices. The shear panels in the Allstate Building were designed to yield at the calculated story shear using the static force procedure.

In addition to this, non-linear three-dimensional analysis was performed on a system model using linear beam elements to model the frames (SH-system), lumped prismatic masses for each floor and a bi-linear two-dimensional hysteresis model for the shear action in each seismic link. A reliability study using 500 earthquake records generated from a modified Kanai-Tajimi spectrum scaled to the local properties of the site was performed and a comparison made to the performance of the previous system and a system with conventional rc-cores assuming linear behavior. Since this study is reported in (4), only the results in terms of standard deviations of the story drifts are given here (Table 1).

System type	dim.	x-direction	y-direction
previous	mm	24.48	30.13
stiff (linear) cores	mm	2.00	1.90
Hyde-system	mm	2.80	2.90

Table 1 Standard deviations of 2nd story drift.

The standard deviation of the story drift is a direct measure of the reliability index as it is defined in modern codes. The comparison shows clearly the effect of the new Hyde-system in the Allstate Building: It limits the standard deviation of the story drifts to stiff-(linear)system values without the large forces. The study also confirmed clearly the inadequacy of the previous system. Given a story failure drift of about 20 mm (elastic drift limit of existing rc-frames), the new Hyde-system provides adequate reliability against this limit state. Thus, extra ductility is not required in the frames of the SH-system and upgrading became obsolete.

4. SUMMARY AND CONCLUSIONS

The seismic retrofit of the Allstate Building in Seattle is presented where a new structural system concept called the hysteretic device - or Hyde-system was used successfully. This new system concept can limit story shear and drift to levels only known from very ductile or very stiff systems respectively thus combining the advantages of both conventional approaches. In the case of the Allstate Building, this rendered upgrading of the existing brittle rc-frames unnecessary resulting in substantial cost-savings and improved performance when compared to conventional systems.

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Seismic Upgrading of an Auditorium Building in Zurich
Renforcement parasismique d'un bâtiment avec auditoires, à Zurich
Erdbebenverstärkung eines Hörsaalgebäudes in Zürich

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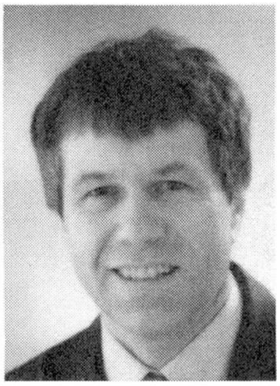

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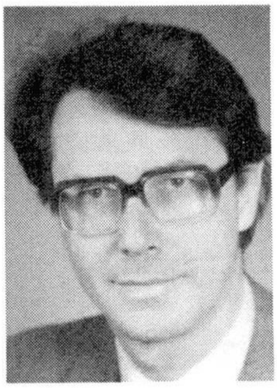

Hugo Bachmann received his Dipl. Ing. and Dr. sc. techn. degrees from ETH Zurich. After practical experience with consulting and contractor firms, he returned to the ETH as Professor of Civil Eng. His and activities concentrate on RC and PC, structural vibrations and earthquake engineering.

SUMMARY

After a new assessment of the overall structural safety of an auditorium building, the seismic safety was found to be insufficient. As an upgrading measure, 30 new diagonal steel truss elements have been placed in the first floor. They support the superstructure and increase the capacity for horizontal seismic forces. The upgraded structure fulfils the current Swiss seismic code requirements.

RÉSUMÉ

La réévaluation d'un bâtiment avec auditoires a mis en question la sécurité parasismique de ce dernier. Le renforcement parasismique du rez-de-chaussée comprend 30 nouvelles colonnes diagonales en acier. Celles-ci améliorent la résistance aux forces sismiques horizontales. Le bâtiment répond ainsi aux exigences de la norme sismique suisse en vigueur.

ZUSAMMENFASSUNG

Im Rahmen der Neubeurteilung der gesamten Tragsicherheit eines bestehenden Hörsaalgebäudes wurde u.a. die Erdbebensicherheit als ungenügend beurteilt. Als Verstärkungsmassnahme sind im Erdgeschoss insgesamt 30 diagonale Stahlstützen eingebaut worden. Diese Stützen verbessern den Tragwiderstand für horizontale Kräfte aus seismischen Einwirkungen. Damit werden die Anforderungen der aktuellen schweizerischen Erdbebennorm erfüllt.

1. INTRODUCTION

The structural safety of a 20 year old university auditorium building with large lecture-rooms had to be reassessed and upgraded.

The building belongs to the university campus of the Swiss Federal Institute of Technology (ETH) at Hönggerberg in Zürich, Switzerland. Figure 1 shows a part of the building during the upgrading construction. The original building's structure before upgrading is characterized by the figures 2 and 3. It has a hexagonal layout with main dimensions of 74m x 69 m. The building's elevation is 22m above and 8m below ground level. The main structural elements are built in reinforced concrete (RC) and in steel.

Originally only the gravity load capacity had to be reassessed. But special features of the structure are a partially soft first storey and an asymmetric configuration of the horizontally stabilizing RC structural walls in this storey. Therefore, beside the vertical gravity loads the horizontal seismic forces had to be included in the structural capacity assessment.

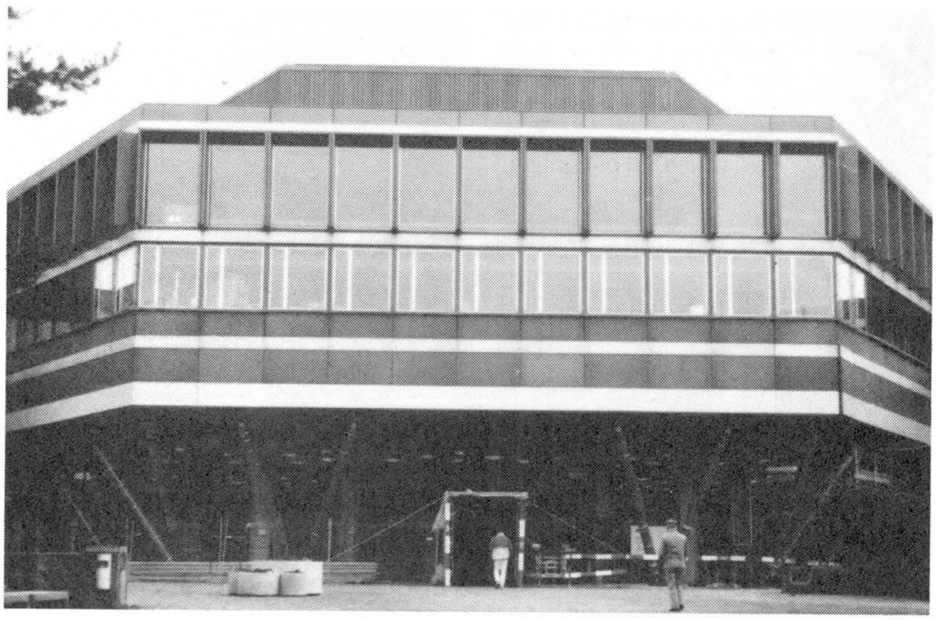

Fig. 1 View from outside towards the upgraded building (during construction). The diagonal steel trusses are the main strengthening elements

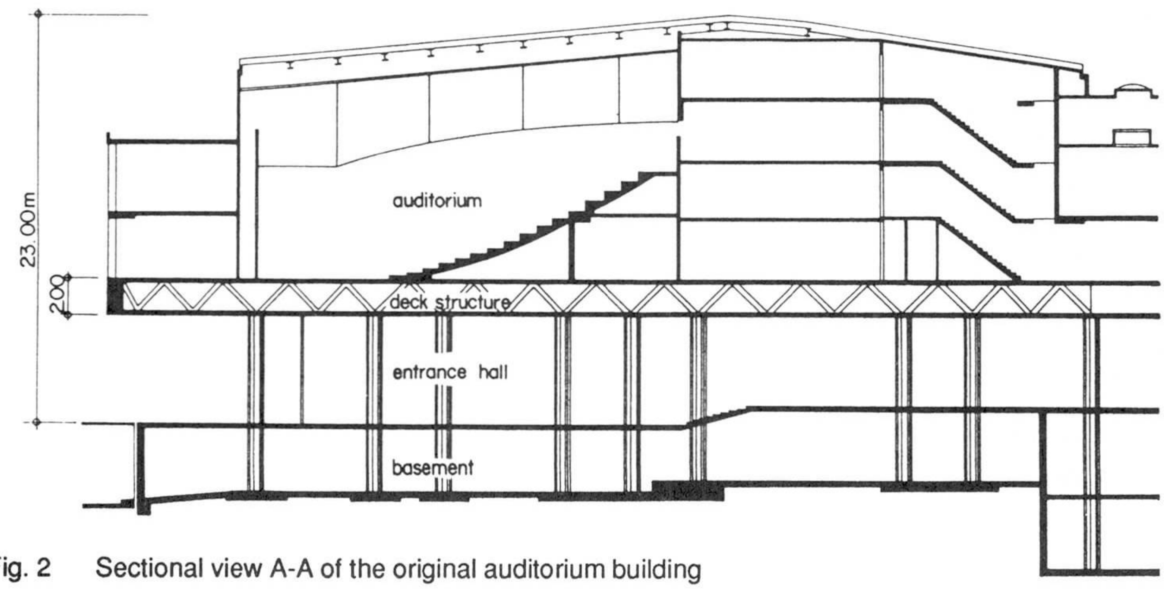

Fig. 2 Sectional view A-A of the original auditorium building

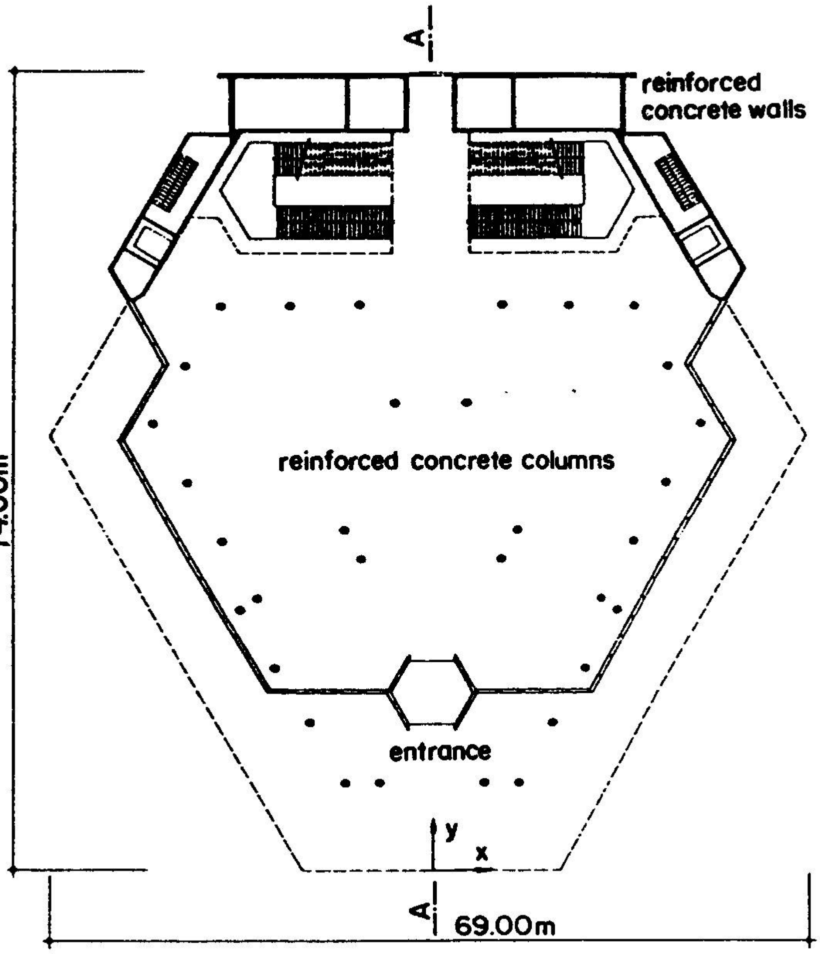

Fig. 3 Plan view of the original auditorium building: first floor

2. SEISMIC ASSESSMENT OF THE ORIGINAL BUILDING

The actual seismic requirements are defined by the Swiss Building Code SIA 160 [1]. The design earthquake has an Intensity of VI – VII (MSK-scale) and corresponds to a recurrence period of 300 – 500 years. It is defined by an effective horizontal ground acceleration of 0.06 g and by a broad-banded elastic design spectrum describing the frequency content of the expected ground motion. For buildings with the importance and the damage potential of the investigated building (building class II) the code allows moderate damage due to design earthquake, but requires sufficient structural capacity to prevent partial or total collapse.

The seismic capacity of the original building was assessed by a combined experimental and analytical investigation. Ambient vibration measurements were carried out to determine the fundamental dynamic characteristics, such as eigenfrequencies and eigenmodes [2]. The design forces were then estimated by a simplified dynamic analysis. The resistance (shear capacity) of the structural system was investigated in a detailed strength analysis.

Figure 4 shows the result of the capacity assessment: the resistance is plotted as a function of the horizontal displacement and compared to the elastic seismic force (seismic design force if a purely elastic response of the structure is assumed). The resistance calculated with help of actual material strengths turned out to be far below these forces. Hence, the code requirements can not be fulfilled by the original structure. Even if large inelastic deformations are tolerated, the available ductility would by far not be sufficient to ensure the building's integrity.

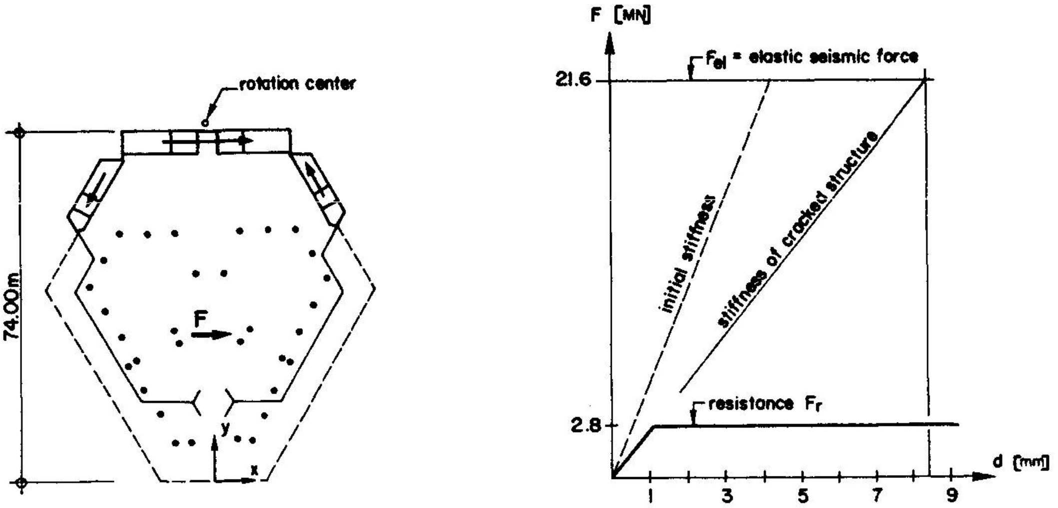

Fig. 4 Resistance to overall seismic forces (x-direction) in the first floor before upgrading (d is the horizontal displacement in the mass center, relative to the soil and foundation)

3. VARIANTS FOR STRUCTURAL UPGRADING

Ten different variants for structural upgrading, including ductile and elastic remaining constructions, were suggested and discussed. Most of them provide additional structural elements which increase the stiffness and the force transfer capacity in the first storey. The two most suitable and efficient variants are described below.

3.1 Variant «Capacity Design»

Figure 5 represents a ductile solution, based on the principles of the capacity design method [3]. The 2 RC concrete blocks with 5 ductile steel elements each, restrained in the concrete block and in the deck structure, provide additional shear capacity in the first floor. The steel elements are designed to act as plastifying link beams.

This solution would serve for seismic upgrading only. For gravity load upgrading additional elements in the hollow space of the deck structure are needed.

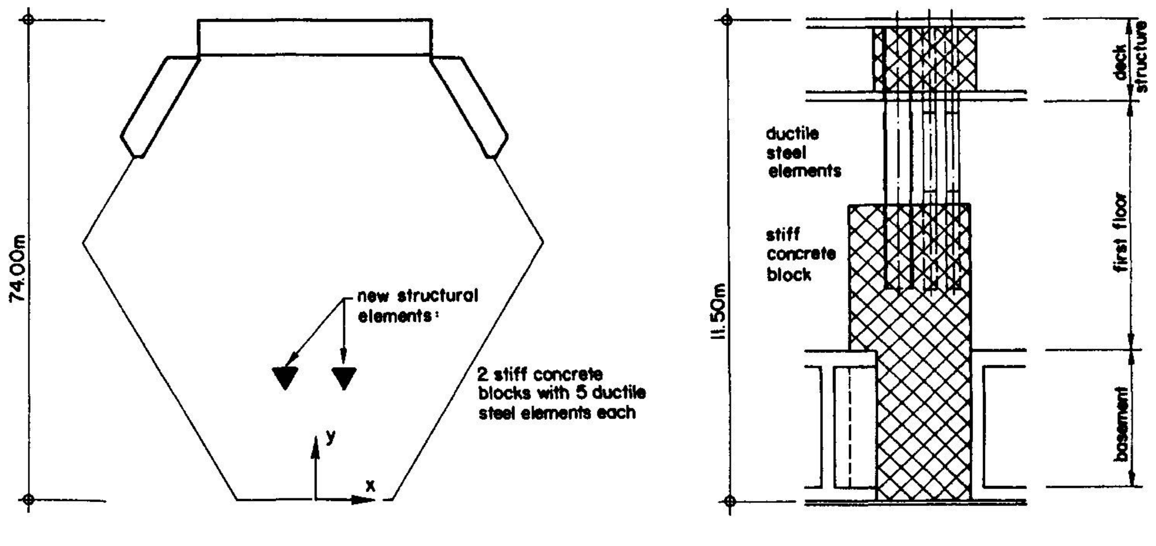

Fig. 5 Upgrading variant "capacity design" with ductile steel elements in a stiff concrete block; left: plan view; right: sectional view

3.2 Variant «Diagonal Trusses»

Figure 6 represents an "elastic remaining" solution serving simultaneously for seismic upgrading and for gravity load upgrading. The solution consists mainly of 30 diagonal steel truss elements placed outside the building to support the deck structure. The truss elements are ring profiles welded at both sides to the joint elements. The support structures at the top and the bottom of the trusses are strengthened by a reinforced concrete girder and a foundation with additional steel anchorage elements (figure 6, right). The foundation of the trusses is integrated in the basement of the building.

The trusses are designed to complement the existing structural walls in the rear part of the building. Simultaneously they enhance the gravity load capacity of the originally cantilevered deck structure.

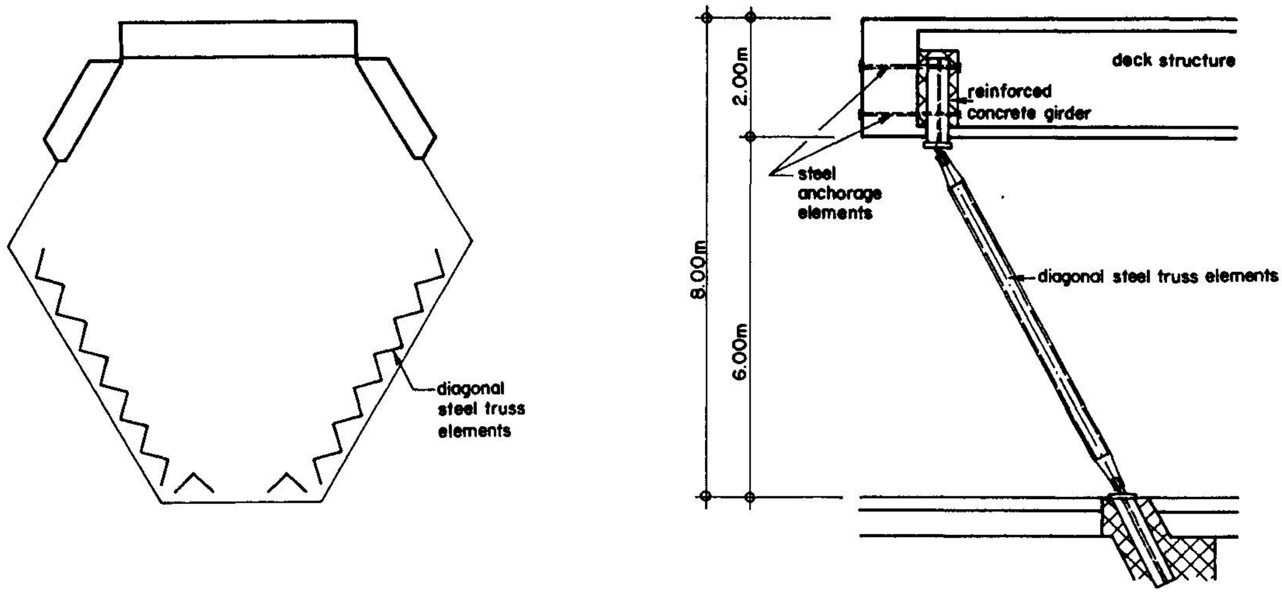

Fig. 6 Realized upgrading variant
left: plan view of new trusses right: sectional view with construction of support strengthening

3.3 Variant Choice and Arguments

After systematic comparisons and discussions between owner, architect, engineer and experts the owner decided to realize the variant «Diagonal Trusses». The main reason for this choice was that the truss elements increase not only the seismic capacity of the building but also the vertical capacity of the deck structure. The cantilever parts with minimal safety margins for vertical loads are significantly upgraded by the new supporting trusses.

With this solution it was possible to realize construction work mainly outside of the building; the lecture activities were not severely disturbed. The extremely tight time schedule for the main construction was limited to the 3-month period of the university's summer vacation 1994.

4. SEISMIC ANALYSIS OF THE UPGRADED STRUCTURE

The new steel trusses are designed to work in the range of their elastic material behaviour, also for the design earthquake. However, the interaction of the original structure with the added truss elements was investigated by a nonlinear static analysis, using the information from the previous measurements and analyses. The structure above the first floor was modeled as a rigid body, stabilized by horizontal elasto-plastic springs representing the original structural walls and the added trusses.
Figure 7 represents the model for the nonlinear static analysis with the finite element code FLOWERS [4] and the resulting force-displacement relationship for excitation in the weak horizontal direction.

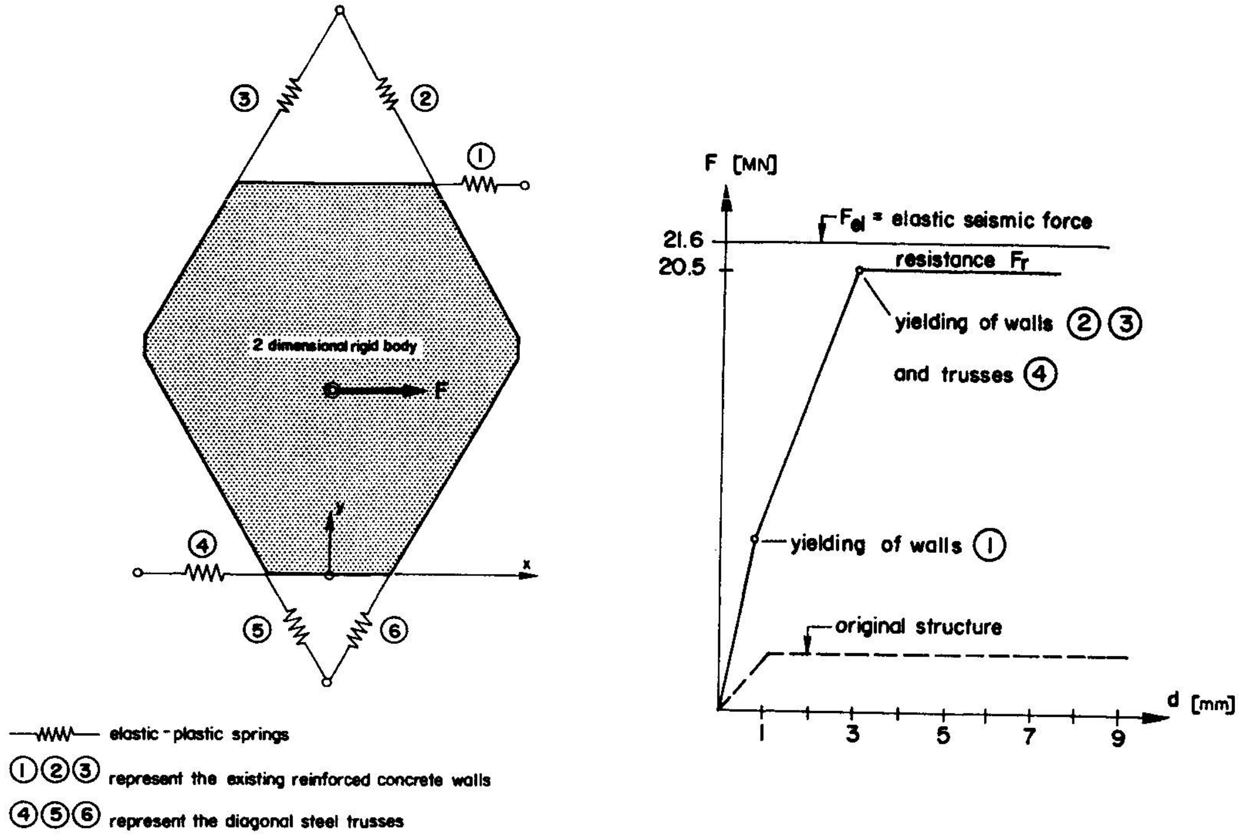

Fig. 7 Analysis model and result of seismic capacity assessment for the upgraded structure

It is obvious that the seismic resistance is effectively increased by the upgrading measures. The resistance is now only very little below the elastic seismic force, determined for purely elastic behaviour of the structure.

Compared to the original structure the first yielding is expected at a significantly higher level of seismic force. The required ductility for the design earthquake is reduced to a maximum ductility factor in the order of 2 - 3 for the RC structural walls in the rear part of the building.

The stiffness is increased by the trusses and the collapse mechanism is improved. When the elastic limits are reached in wall 1, the forces can be further increased and rearranged to the remaining structural elements. Under the design earthquake the total displacements in the mass center are limited to approximately 3 mm, relative to the soil and foundation.

5. CONCLUSION

The upgraded building fulfills the seismic requirements of the Swiss Building Code adequately.

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Hidden Lateral Strength in Older Homes

Résistances latérales cachées dans les anciennes maisons

Versteckte Aussteifungen in älteren Häusern

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SUMMARY

This paper investigates the hidden lateral strength in older homes. It begins by reviewing current knowledge of residential seismic performance. A brief discussion of the United States' Uniform Building Code definition of conventional light wood-frame construction is included. This type of construction is analysed through computer modelling to determine the theoretical strength of older frame construction walls. Additional strength is identified through configuration of the home. Finally, recommendations for future code consideration are presented to improve the performance of homes in high seismic zones.

RÉSUMÉ

L'article étudie les résistances latérales présentes dans les anciennes maisons. Il passe en revue l'état des connaissances sur la performance sismique des habitations. La définition de la construction légère classique utilisant des charpentes en bois est donnée d'après le "Uniform Building Code" des États-Unis. Ce type de construction est alors étudié à l'aide d'un modèle informatique, pour déterminer la résistance théorique des murs construits avec les anciennes charpentes. L'analyse met en évidence une résistance supplémentaire, dépendant de la configuration de la maison. Des recommandations sont faites afin d'améliorer les performances des habitations dans les zones à haut risque sismique.

ZUSAMMENFASSUNG

Diese Abhandlung untersucht die versteckten Aussteifungen in älteren Häusern. Sie beginnt mit einem Rückblick auf das bisher bekannte Wissen über die Widerstandsfähigkeit von Wohnhäusern gegen erdbebenbedingte Erschütterungen. Sie beinhaltet auch eine kurze Betrachtung der Definition des Uniform Building Code der USA. für konventionelle Leicht-Holzbalkenkonstruktion. Diese Konstruktionsart wird durch Computermodelle analysiert, die den theoretischen Widerstand älterer Wände in Balkenkonstruktion bestimmt. Zusätzlich wird der Widerstand in Abhängigkeit von der Bauart der Häuser beschrieben. Abschliessend werden Empfehlungen für zukünftig zu berücksichtigende Werte gegeben, um den Widerstand von Häusern in erdbebengefährdeten Gebieten zu verbessern.

1. INTRODUCTION

Residential construction has not been a recent priority for seismic research. While studies have been done to define strength parameters for new construction, little recognition has been given to older home construction and its hidden strengths. For years, the United States' Uniform Building Code (UBC) has recognized that older homes have inherent lateral strength through the provisions of Section 2517, unofficially referred to as the "prescriptive method" of lateral analysis. This section defines code provisions for conventional light wood-frame construction. Specific definitions are given to different bracing systems and the configuration of the structure. If falling within Section 2517's provisions, the house is deemed acceptable for most seismic and wind zones. This paper will present a review of residential home performance in earthquakes, it will study how the configuration provisions of 2517 provide a stronger-than-expected home, and it will make recommendations for residential seismic code improvements.

The general public does not realize that most building codes are primarily focused on life-safety. In a disaster such as an earthquake, the UBC's primary goals are to resist collapse and allow safe exit of the building occupants. Little consideration is given to the amount of structural and non-structural damage sustained by the building. After the January 17, 1994 magnitude 6.8 Northridge, California earthquake, 15,000 homes and apartments were made uninhabitable. Tens of thousands of additional homes were sufficiently damaged to require costly repairs. Is this acceptable in a country with stringent building codes and regulations? Physical, mental, and economic damage to a community after a disaster can be devastating. We have the knowledge in our engineering community to mitigate much of this residential damage through recognition of older homes' hidden strengths. If we integrate these ideas more closely into our building codes in high seismic zone areas, the savings to homeowners, the community, insurance companies, and the government can be tremendous.

2. RESIDENTIAL SEISMIC PERFORMANCE - LITERATURE SEARCH

The Earthquake Engineering Research Institute publishes "Earthquake Spectra," one of the best sources of findings after each major earthquake. Often there is little information about the seismic performance of homes. Most often mentioned reasons for damage are:

1. The lack of a continuous load path from roof to the foundation.
   a. The lack of anchor bolts between the house and foundation.
   b. Unbraced cripple walls between the house and foundation.
   c. The lack of properly constructed shear-resisting walls.
   d. The lack of a proper method to resist shear wall overturning.
2. Lack of bracing/strapping for the hot water heater.
3. Building geometric and stiffness irregularities.
4. Precarious site conditions, such as liquefaction and steep slopes.

Figure 1 - Typical Early 1900's Home

While "Earthquake Spectra" reports damage findings, few reports have studied what types of homes have performed well in earthquakes. Good design, construction, and inspection practices can address all of the above deficiencies, except for site limitations. With the techniques identified in this paper, additional strength can be designed into homes without increasing construction costs. To understand the importance of configuration in residential lateral design, it is important to first understand some of the conventional light wood-frame provisions in the UBC's Section 2517. Figure 1 shows a typical older home conforming to these provisions.

3. CONVENTIONAL LIGHT WOOD FRAME CONSTRUCTION - UBC LATERAL DESIGN GUIDELINES

Conventional light wood-frame construction is the typical type of construction employed in the timber framing of most homes in the United States. The 1991 Uniform Building Code, Chapter 25, Section 2517, Conventional Construction Provisions, defines design guidelines for conventional light wood-frame construction. This section also refers to a Table No. 25-V - Wall Bracing. This table presents acceptable wall bracing provisions for the four seismic zones, and is referred to as the "prescriptive method of lateral design". From a code interpretation in the Building Standards magazine, Section 2517 and Table 25-V are limited "to regular, or conventional, structures... In general, regular structures have no significant discontinuous elements in plan or elevation and the lateral force-resisting system is positioned parallel to the major orthogonal axes. Regular structures in plan are without reentrant corners associated with "L"- or "T"-shaped systems; roof and floor diaphragms are without abrupt discontinuities or large openings; there are no out-of-plane offsets in vertical bracing elements; and bracing elements are uniformly distributed parallel to the major axes of the structure. Regular structures in elevation are without structural discontinuities such as in-plane offsets of bracing elements and large mass or geometric differences between stories or levels. If the framing member sizes of a light-frame structure are selected in accordance with Section 2517 and the bracing system is without offsets in both the horizontal and vertical planes, the conventional construction provisions are applicable in Seismic Zones Nos. 2, 3, 4."[2] Figure 4 shows Table 25-V as presented in the 1991 Uniform Building Code.

The historical significance to this section is important because it is the nearest the UBC comes to recognizing the hidden lateral strength of older homes. The Handbook To The Uniform Building Code gives some explanation as to the origin of these prescriptive provisions. The last modification to the wall bracing requirements in Section 2517 and Table 25-V came after the 1971 San Fernando earthquake. As the Handbook explains, "The provisions of Section 2517 are based on experience gained over the last 60 years or more."[3] Though the Handbook doesn't elaborate, most likely this experience was based upon observations of older homes (early 1900's), among others. Examining Figure 5 (25-36 from the Handbook), the sketches look conspicuously like many of the popular older home styles. The blank areas between the cross-hatching would typically represent windows or doors. Among others, homes from the French architecture (Second Empire, French Eclectic), the English architecture (Georgian, Adam, Colonial Revival), and Italian architecture (Italianate, Italian Renaissance) falls into the 25-V category. All of these styles are prevalent in older American cities throughout the United States, as well as throughout Europe. One can surmise that this style of regularly spaced walls defined in 25-V occurred because glass was scarce and expensive in early homes. Therefore, windows were located at the center of rooms to maximize natural lighting. The window positioning typically happened to leave 4 foot (1.2 m) wide walls in the corners and regularly spaced 4 foot (1.2 m) panels throughout the home.

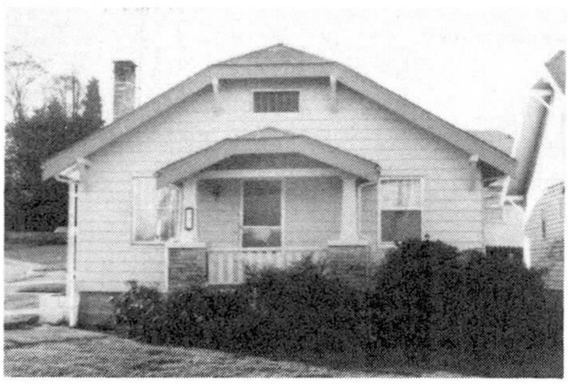


Figure 2 - A Typical Prescriptive One-Story Home

Figure 3 - A Typical Two-Story Prescriptive Home

TABLE NO. 25-V—WALL BRACING

SEISMIC ZONE	CONDITION	TYPE OF BRACE[1] A	B	C	D	E	F	G	H	AMOUNT OF BRACING[2]
0, 1 and 2	One Story Top of Two or Three Story	X	X	X	X	X	X	X	X	
	First Story of Two Story or Second Story of Three Story	X	X	X	X	X	X	X	X	Each end and each 25′ of wall
	First Story of Three Story		X	X	X	X[3]	X	X	X	
3 and 4	One Story Top of Two or Three Story	X	X	X	X	X	X	X	X	Each end and each 25′ of wall
	First Story of Two Story or Second Story of Three Story		X	X	X	X[3]	X	X	X	Each end. 25% of wall length to be sheathed
	First Story of Three Story		X	X	X	X[3]	X	X	X	Each end. 40% of wall length to be sheathed

[1]See Section 2517 (g) 3 for full description.
[2]Bracing at ends shall be near thereto as possible. Braces shall be installed so that there is no unbraced section along the wall exceeding 25 feet.
[3]Gypsum wallboard applied to supports at 16 inches on center.

A - 1"x4" Let-In Braces
B - 5/8" Minimum Diagonal Sheathing
C - 3/8" Minimum Plywood Sheathing
D - 1/2" Minimum Fiberboard, 4'x8' Sheets
E - 1/2" Minimum Gypsum Wall Board
F - Particle Board Wall Sheathing Panels
G - Portland Cement Plaster
H - Hardboard Panel Siding

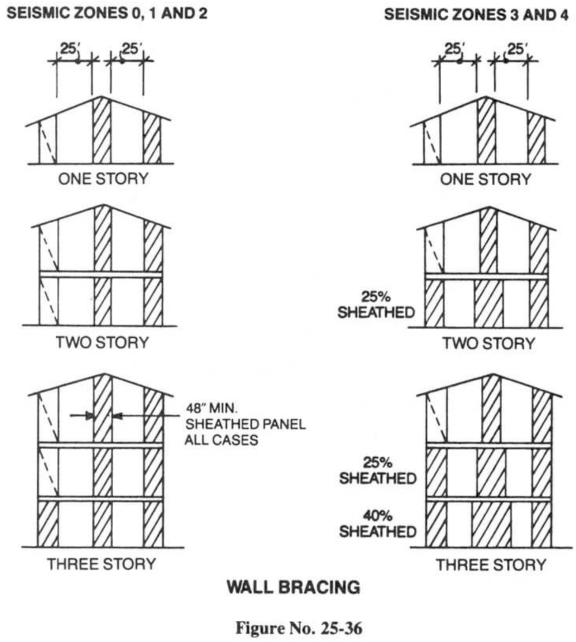
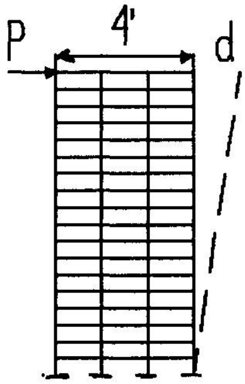


Figure 4 UBC Table 25-V [4]

Figure 5 Handbook to the UBC Figure 2 5-36 [3]

4. ANALYTICAL STUDY OF TRADITIONAL HOME DESIGNS

The 4'(1.2m) wide wall panels are identified as key to the UBC prescriptive method. These can easily be seen in the pictures, UBC Table 25-V, and the UBC Handbook 25-36. These 4'(1.2m) panels were typically thought to act in shear. Apparently ignored was the approximately 2'(0.6m)-4'(1.2m) wide beams connecting the 4'(1.2m) panels. Since these panels are typically skip-sheathed, this investigation focused on the whole wall acting more flexibly as a portal frame. First, the strength of a typical skip-sheathed 4'(1.2m) wall panel had to be quantified. A finite element frame model was created as shown in Figure 6. The model is a 4 foot (1.2 m) wide wall, with heights varying between 7 feet (2.4 m) and 10 feet (3.0m) high. The verticals are 2"(5 cm)x4"(10 cm) Douglas Fir members, spaced at 16" (40 cm) on center. The horizontals are 1"(2.5 cm)x6"(15 cm) tongue-in-groove skip sheathing on both the inside and outside walls. The horizontals are connected to the verticals with 2-8d nails spaced 5"(13 cm) apart. The limiting factor for loading this frame is the nail connection. The maximum nail allowable shear is 129 pounds (574 nt). The maximum beam connection moment for the element, then, is figured using a 5"(13 cm) moment arm, per side. The maximum panel shear load was iterated until the first element connection reached its maximum allowable moment. The resultant loads and corresponding deflections are shown in Figure 6 below. In this case, a linear elastic analysis is valid because loads and deflections are so low.



7'(2.1m) Wall Height, P = 1,160#(5,162nt), d = 0.026"(0.07cm)

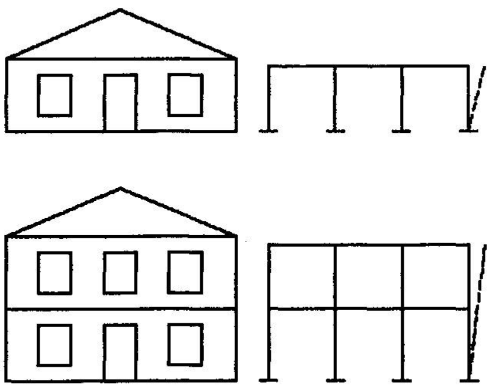
8'(2.4m) Wall Height, P = 1,148#(5,109nt), d = 0.030"(0.08cm)

9'(2.7m) Wall Height, P = 1,140#(5,073nt), d = 0.036"(0.09cm)

10'(3.0m) Wall Height, P = 1,136#(5,055nt), d = 0.042"(0.10cm)

Figure 6 - Frame Analysis Results For 4' Panel

After the frame panel analysis was complete, equivalent wall properties were determined to model a simple frame for a side of both a one-story and two-story house of varying wall heights. A typical house configuration was selected as shown in Figure 7. The house is 28'(8.5 m) square in plan, with a 6'(1.8) high roof. The model is shown in Figure 7with each member modeled as a skip-sheathed panel from the first analysis. Note the difference between our model and the 25-36 sketches of Figure 5. The portion crucial to our model, and ignored in the prescriptive method, is the beam section of the portal frame. To ensure that compatibility was maintained with the results of the single 4'(1.2 m) panel, story drift was limited to the deflection of the single 4'(1.2 m) panel, 0.031"(0.8 mm). The resultant loadings are shown in Figure 7. These loads are commensurate with Zone 4 seismic loads as defined in the UBC.



	7'(2.1m)	8'(2.4m)	9'(2.7m)	10'(3.0m)
P	4400#	4400#	4400#	4400#
d	0.026"	0.030"	0.036"	0.042"
P2	2600#	2700#	2800#	2900#
d2	0.052"	0.060"	0.072"	0.084"
P1	1300#	1200#	1100#	1000#
d1	0.026"	0.030"	0.036"	0.042

Figure 7 One- and Two- Story Frame Model Results

5. CONFIGURATION: THE ADDITIONAL HIDDEN STRENGTH OF OLDER HOMES

Though limited, the above analysis shows that older homes are significantly stronger than previously thought. The prescriptive 4'(1.2 m) panels spaced at regular intervals are very strong. But there are other hidden and unquantified areas within older homes that contribute further strength. Those areas we lump into the term configuration:

1. Redundant load paths. Beyond the regularly spaced 4'(1.2 m) panels, most older homes hae small rooms and many interior walls. These interior walls form secondary (redundant) load paths, providing extra strength
2. A lightweight structure, resulting in lower seismic loads.
3. Stronger framing lumber. The structure is composed of old growth, strong, full-dimensional lumber.
4. A symmetric structure, symmetric about both axes.
5. A continuous load path. Load paths are continuous from roof to foundation. Structure/foundation attachment, while not mechanical, often had an of embedded the bottom sill into the concrete foundation wall.
6. A flexible structure with high damping characteristics, due to the many nailed connections and friction of the tongue-in-groove skip sheathing.
7. Secondary strength contributors, such as the interior sheetrock or plaster and exterior siding on older homes.

With properly connected interior walls, 50% redundant capacity can be easily shown. The author estimates that these configuration factors can contribute 50%-100% additional lateral load capacity.

6. RECOMMENDATIONS FOR RESIDENTIAL CODE IMPROVEMENTS

Residential prescriptive codes should be improved to include the importance of configuration, including:

1. A continuous load path.
2. A symmetric structure, without geometric or stiffness discontinuities.
3. A relatively lightweight, flexible structural system with high damping.
4. A prescriptive approach to lateral design.
5. A secondary/redundant load path.

Home designers, engineers, architects, and building officials should recognize the importance of the UBC Section 2517 provisions for prescriptive lateral design, and the importance of configuration. Building codes should mandate that home designs in seismic Zones 3 and 4 fulfill the intent of these characteristics. Proper designs can implement all of these characteristics without increasing construction costs. Additional research beyond this paper is needed, but the importance of the prescriptive approach, along with thoughtful configuration design, has been demonstrated.

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Strengthening of an Unreinforced Masonry Building
Renforcement d'un immeuble en maçonnerie non armée
Verstärkung eines unbewehrten Mauerwerkbaus

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SUMMARY
50 Green Street is a historic brick building located in San Francisco's warehouse district. Originally built for warehouse and manufacturing use, the massive structure possessed many of the deficiencies common to unreinforced masonry construction. The strengthening scheme was sensitive to the architectural fabric of the building and included steel knee-braced frames, steel tension-rod roof diaphragm strengthening, and a pioneering effort incorporating the use of "CenterCore" reinforcement in the existing brick walls. The owner was actively involved in the process and his special concerns for cost, time, appearance, and tenant disruption were incorporated as the system evolved.

RÉSUMÉ
Erigé à l'origine pour servir de bâtiment d'entrepôt et de manufacture, cet ouvrage massif présente les nombreux défauts des constructions en maçonnerie non armée. Le concept de renforcement prenait en compte la structure architecturale de l'immeuble. Il comportait des cadres en treillis d'acier articulés, des barres de traction pour renforcer les fermes de toiture et l'incorporation d'armatures spéciales appelées "CenterCore" dans les murs en maçonnerie existants. En étroite collaboration avec le maître d'ouvrage, les travaux de rénovation furent menés à bien en tenant compte des considérations de coûts, de temps, d'aspect extérieur et de perturbation des locataires.

ZUSAMMENFASSUNG
Ursprünglich als Lager- und Fabrikgebäude errichtet, weist das massive Bauwerk viele Mängel üblicher unbewehrter Mauerwerksbauten auf. Das Verstärkungskonzept nahm Rücksicht auf die architektonische Struktur. Es beinhaltete K-Fachwerk-Stahlrahmen, Zugstangen als Verstärkung der Dachscheibe und die Verwendung von sog. "CenterCore"- Bewehrung in den bestehenden Mauerwerkswänden. In enger Zusammenarbeit mit dem Eigentümer wurden seine Anliegen betreffend Kosten, Zeit, Aussehen und Störung der Mieter in das Sanierungskonzept eingearbeitet.

1. INTRODUCTION

Whenever new construction involves work on an existing structure, special considerations will arise that require the designer to be imaginative in the application of strengthening principles, and flexible in the implementation of strengthening efforts. This is especially true in the case of unreinforced masonry buildings. Often these structures have designated historical significance or a sentimental attachment to the community in which they reside. These conditions can limit the extent to which the structure may be modified when making seismic improvements.

50 Green Street is a two-story brick structure located north of Market Street in San Francisco's warehouse district. It occupies the entire city block between Green, Commerce, Battery, and Front Streets. The exterior facade consists of an arcade of slender piers and graceful semicircular arches along all four elevations. Ornate brick relief patterns around the arches, raised detailing in the four corners, and corbelled courses of brick at the parapet give the building a distinctive appearance.

Fig. 1 Building Exterior

Construction on 50 Green Street began in early 1906, and was interrupted by the San Francisco earthquake and fire in April of that same year. Situated in an area devastated by the fire, the building was reconstructed using the original plans, and completed in 1907. Originally built as the W.P. Fuller & Co. Glass Warehouse, unique features included a railroad spur that entered the east end of the building, and enlarged arches in the center of the north and south walls for a drive through. Current use includes upscale office space for advertising and movie industry tenants.

2. DESCRIPTION

50 Green Street is a two-story unreinforced brick masonry bearing wall structure with a full basement. It is rectangular in plan, measuring 37 meters by 84 meters. The overall height is about 15 meters from basement to top of parapet, with a first story of 7 meters. Exterior walls vary in thickness from 71 cm at the first floor arched piers, to 33 cm at the parapet. Two interior 43 cm brick firewalls divide the building into three unequal areas. All walls below grade are concrete, and are founded on concrete spread footings. Including the storage areas that comprise the basement, the building has approximately 9,200 square meters of usable space.

The building was designed for heavy vertical loading associated with manufacturing and warehouse use. The floors consist of two layers of structural planking on closely spaced wood joists. The joists lap over the top of heavy timber girders spanning between massive interior knee-braced columns. Heavy timber trusses form the pitched roof structure.

3. DESIGN CONSIDERATIONS

Although unreinforced masonry buildings (UMB's) have been the target of recent legislation requiring mandatory seismic strengthening, evaluation and strengthening of 50 Green Street was commissioned by the building owners in 1991, before local UMB ordinances were finalized.

The U.C.B.C. Appendix Chapter 1, "Seismic Strengthening Provisions for Unreinforced Masonry Bearing Wall Buildings" was originally selected as an appropriate strengthening criteria, with a specified base shear coefficient of 0.133g. This document was the model for the Draft S.F. UMB Ordinance which was finalized and adopted in 1993 as Chapters 14 and 15 of the San Francisco Building Code. Subsequent redesign requested by the owner to accommodate tenant concerns utilized the S.F. UMB Ordinance with a lower base shear coefficient of 0.10g.

The UMB Ordinance contains a set of minimum standards designed to reduce, but not necessarily eliminate, the potential hazards common to most unreinforced masonry buildings. Hence they are considered "hazard reduction" measures. It is important that the owner understands the potential for damage during a major earthquake still exists, even after strengthening work is completed. The ordinance prescribes allowable capacities for existing brick walls and wood diaphragm assemblies, as well as new anchors installed into existing brick. It also specifies allowable wall slenderness ratios to address out-of-plane stability. Other provisions include mandated wall anchorage for out-of-plane forces, parapet bracing, and supplemental vertical support for truss and girder elements in case the bearing walls lose integrity.

The work required some testing and research. A geotechnical investigation was performed to determine the condition of the foundation and soil design values. In-situ brick shear testing was performed to determine the quality brick masonry construction. Mortar bond strengths tested well over 690 kPa. With the owner's concerns for minimizing disruption in mind, research into alternative methods of strengthening led us to consider the patented "CenterCore" method of wall reinforcement, which involves installing reinforcing bars into grouted cores drilled vertically through the walls. Full scale testing of this system performed at California State University Long Beach in the early eighties, as reported by Breiholz in 1987, demonstrated the effectiveness of CenterCores for improving the in-plane shear strength and out-of-plane stability of brick walls. These test results were utilized at 50 Green Street.

4. OWNER AND TENANT IMPACTS

The owner was actively involved in the decision making process as his special concerns for cost, time, appearance, and tenant disruption were incorporated as the strengthening scheme evolved. A major redesign of steel concentric braced frame elements proposed for the longitudinal walls was requested when the owner learned that one major tenant was demanding significant concessions for the disruption caused by the work in their space. The concept was changed to a knee-braced configuration to avoid impacting the arched windows, and the frame elements were shifted towards one end of the building to lessen the impact on the tenant space. The new configuration had the added architectural benefit of mimicking the knee-braced framing of the existing floor construction.

Another concern was a new roof membrane that was recently installed. The owner wanted to leave it intact, so roof diaphragm strengthening was restricted to inside the building, and conventional plywood sheathing was not an option.

Finally, it was crucial to the owner that he maintain his current tenant base, so the work had to be completed while the building was fully occupied. A phased construction schedule was developed by the contractor for work performed during nights, weekends and holidays. At the end of each weeknight shift the building was returned to the tenants the next morning. This dramatically extended the duration of construction and resulted in a monumental clean up effort by a special janitorial crew each day.

5. SCOPE OF WORK

The massive nature of the construction at 50 Green Street, so strong for vertical loads, was actually contributing to the seismic deficiencies of the building. Code prescribed capacities for brick walls and wood diaphragms were not adequate for the seismic forces generated in the building. It was an interesting challenge to point out walls, 71 cm thick, and heavy timber framing to an owner who is very proud of his building, and attempt to explain just how "weak" they can be. In-plane wall strengthening was required in both directions, and diaphragm strengthening was required at all levels. The thick exterior walls satisfied UMB Ordinance slenderness requirements, but the thinner interior walls required out-of-plane stabilization.

A steel knee-braced frame was selected to supplement the longitudinal walls. A relative rigidity analysis was performed using the structural analysis program, RISA-2D, to verify that the knee-braced system would draw load from the existing brick walls. The resulting W36x150 column members are stiffness controlled. They are embedded in a concrete foundation wall that extends up to the sill of the arched windows, providing a fixed base and shortened effective story height. Centered on the arched piers, the new columns must support the existing floor girders that frame into the wall at the same location. The girders were shored, cut back from the wall to erect the columns, and then seated on a bracket welded to the column web.

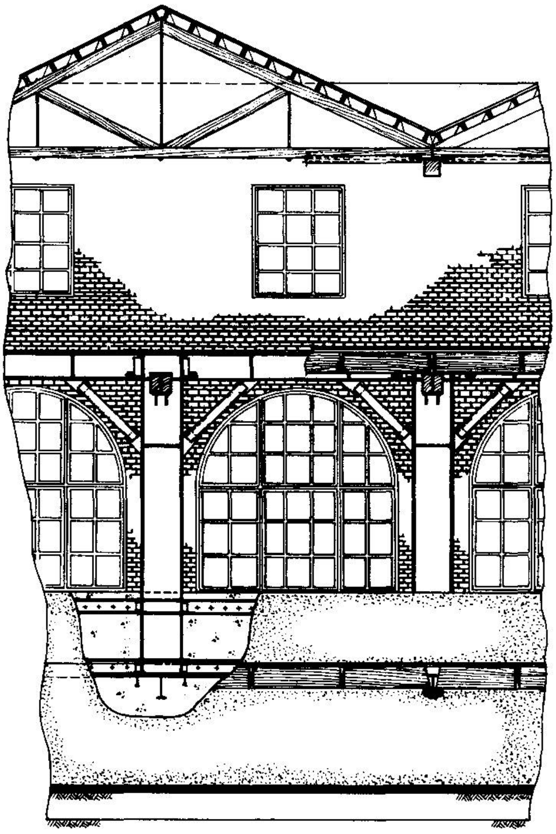

Fig. 2 Drawing of Knee-Braced Frame System

The most critical connections occurred along the length of the drag strut which delivered load to the knee-braced frames located at one end of the building. The drag consists of a large tube section running along the wall just below the floor joists. At each pier the existing wood girders interrupted the continuity of the drag. Horizontal slots were drilled though the girders to allow splice plates to pass through and weld to the tube on either side. The drag strut also served as a horizontal strong-back, anchoring the walls to the heavy floor girders for out-of-plane loads. Diaphragm to wall connections were made using threaded epoxy anchors, installed after the drag strut was in place. The anchors were drilled to within 5 cm of the exterior surface of the wall, and were responsible for both in-plane shear transfer and out-of-plane wall anchorage.

In the transverse direction, a more favorable pier configuration existed, allowing the use of the existing brick walls for transverse lateral force resistance. To strengthen the walls, deformed reinforcing bars were installed using the CenterCore technique. Cores were added at wall locations that were highly stressed for in-plane shear, and at window and door jambs to provide trim reinforcement wherever possible. CenterCores provided reinforcement to stabilize the slender interior walls for out-of-plane forces, and were designed using standard reinforced masonry principles.

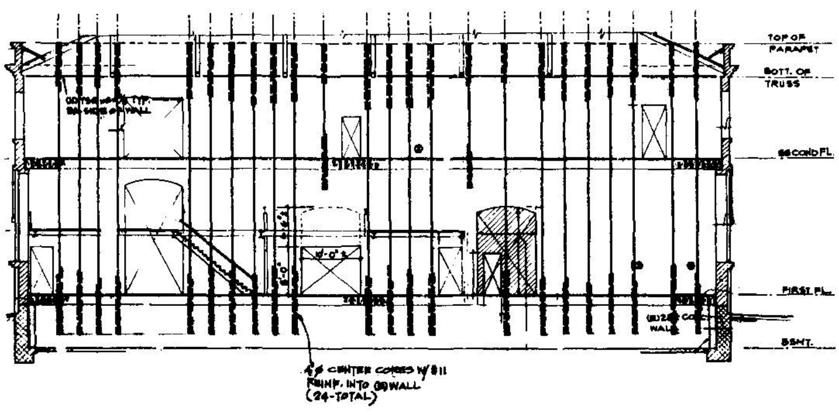

Fig. 3 Interior Elevation of CenterCore Locations

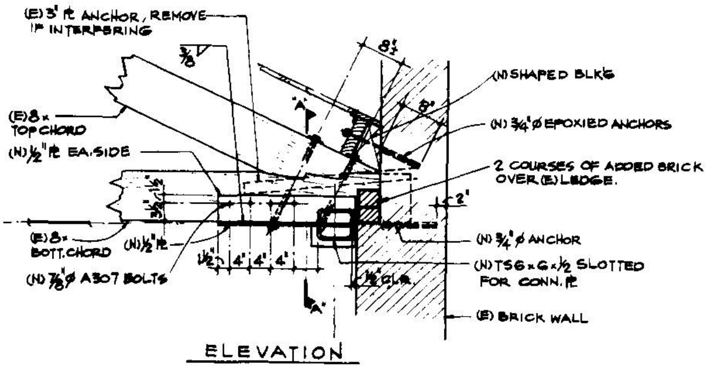

Fig. 4 Roof Diaphragm Connection at Truss

The CenterCore technique involves core drilling vertically through the wall from parapet to foundation, and installing grouted reinforcing bars. Coring specifications allowed a tolerance of two inches out of plumb at the base of the wall, 15 meters below the top of the parapet. Occasional repairs to wall surfaces were required when the bit broke through the side of the wall after being forced off line by unexpected iron embedded in the wall. A dry coring method was selected to avoid the disruption of water associated with wet coring operations. This resulted in dust migrating through micro-cracks in the walls during coring operations. Dust became so severe that the entire length of each wall within the building was wrapped in plastic and ventilated with negative air machines to prevent infiltration throughout the tenant spaces. Based on lower unit cost and superior performance noted in the Breiholz report, polyester based resins were specified for grouting the cores. The resin emanated such a strong styrene odor, however, that tenant complaints forced a change to a more expensive epoxy based resin, that had comparably little lingering odor.

To avoid the new roof membrane, a steel tension rod diaphragm was installed at the bottom chord of the roof trusses. Slotted tube connections, clevises, and turnbuckles allowed fit up tolerance and angular adjustments during erection. In anticipation of inevitable variations in field conditions, a liberal safety factor was incorporated into the design. This came in handy when existing conditions were not "square" and eccentricities had to be built into the system. Particularly challenging, were connections between the new steel elements and the integral wood trusses. Connection brackets were prefabricated, and consisted of vertical tabs for bolting into the side of the truss, welded to horizontal gusset plates. Extra bolt holes were provided in the tabs in case non-typical truss connections interfered with a prefabricated bolt location.

As a diaphragm, the sturdy floor system was also overstressed by seismic forces generated by the massive walls. Plywood diaphragm strengthening was provided over the most highly stressed regions around the perimeter. Since the tenant spaces were to be returned by the start of each work day,

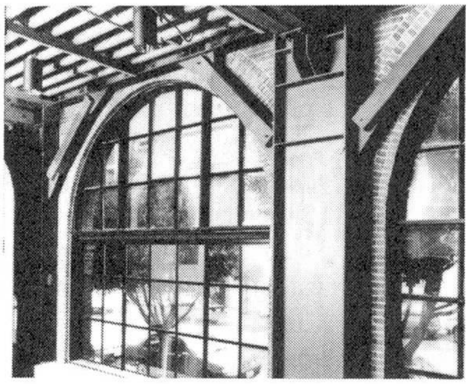

Fig. 5 Completed Knee-Braced Frame

plywood was installed in small sections. Offices were cleared out on Friday afternoon, sheathed in plywood, inspected, carpeted, and moved in by Monday morning.

The result of this work was a strengthening system that provided a complete load path for seismic forces which was sensitive to the architectural fabric of the building. The completed work was recognized by the Foundation for San Francisco's Architectural Heritage, and received their Award for Excellence in Architectural Preservation.

6. TIMELINE AND COST IMPACTS

The original construction documents were completed in 1992, and estimates for construction costs totaled about $2,800,000. Subsequent redesign to address tenant concerns was completed in 1993, with a bid for construction costs of roughly the same amount. Phased construction began in September of 1993 with substantial completion in May of 1994, eight months later. It is estimated that phased construction during off hours in the fully occupied building increased the cost of this work by 25% over that in an unoccupied building. Special janitorial services totaled $50,000. Final costs, after field change orders, totaled about $2,900,000, within 3% of the original bid. Close cooperation with the contractor, and sensitivity to cost and constructablity of changes, kept this difference to a minimum. The cost of CenterCoring operations was estimated to be $150 per foot of core, including drilling and grouting. Because of out-of-plane problems associated with the interior transverse walls, CenterCore strengthening was estimated to be roughly the same cost as conventional strengthening for these walls, but had the added benefit of less disruption and no architectural impact.

7. ACKNOWLEDGMENTS

Contributions from Don Davella of Plant Construction Company regarding detailed cost information and the specifics of tenant impacts on construction are gratefully acknowledged.

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Repair and Consolidation of the Masonry Structure of a Building

Réparation et consolidation de la structure d'un bâtiment en maçonnerie

Sanierung und Verstärkung der Mauerwerkskonstruktion eines Gebäudes

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SUMMARY

The paper presents a study of a case about the damages caused by the 1977 earthquake to the structure of a building in Craiova, Romania, as well as repairing and consolidation solutions for this structure. The proposed solutions intend to restore the initial bearing capacity of the structure and its safety.

RÉSUMÉ

L'article porte sur le dégâts provoqués par le séisme de 1977 sur la structure d'un bâtiment à Craiova, Roumanie, ainsi que sur les solutions proposées pour sa réparation et son renforcement. Les solutions proposées tendent à rétablir la résistance initiale du bâtiment ainsi que la sécurité dans son exploitation.

ZUSAMMENFASSUNG

Diese Arbeit berichtet über die Schäden, die am Gebäude der Fakultät für Landwirtschaft von Craiova, Rumänien, infolge des Erdbebens vom 1977 aufgetreten sind. Auch einige Sanierungsmassnahmen wurden vorgeschlagen. Diese Lösungen sollten die ursprüngliche Tragfähigkeit und Sicherheit der Konstruktion wiederherstellen.

1. INTRODUCTION

Statistical analyses of the damage in masonry buildings show that the damage depends mostly on the quality of the materials used and on the quality of the construction, and only a little on the structural geometry of the building. However, the practical experience demonstrates that, during violent earthquakes, the conception of structural geometry is very important for the building survival.

Thus, it is very important weather the plane form of the masonry structure is regular or irregular, symmetrical or non-symmetrical and weather the building has a pronounced asymmetry of the volume, mass and rigidity distribution or not. In the same time, it is very important weather the building has one or two storeys or it is a multi-storey building.

In general, the seismic resistance of the masonry buildings is assured mainly by a number of large shear walls, in each principal horizontal direction, that are able to support the most important damages during the earthquake.

The correct design of the new masonry structures supposes not only an adequate calculus, but also the adoption of the constructive measures that will give to the structure an increased security against seismic actions, ensuring the structure's survival and the avoidance of causing victims and exaggerated damages.

The old buildings structures were not designed to resist to violent earthquakes because at the time they were designed and erected, the problem of antiseismic design and measures for the buildings was not properly considered and there were any norms or instructions for the antiseismic protection of the structures.
Therefore, in the world there is a large number of such old buildings, damaged during the earthquakes, that no longer present security in strong seismic loads and that have become a

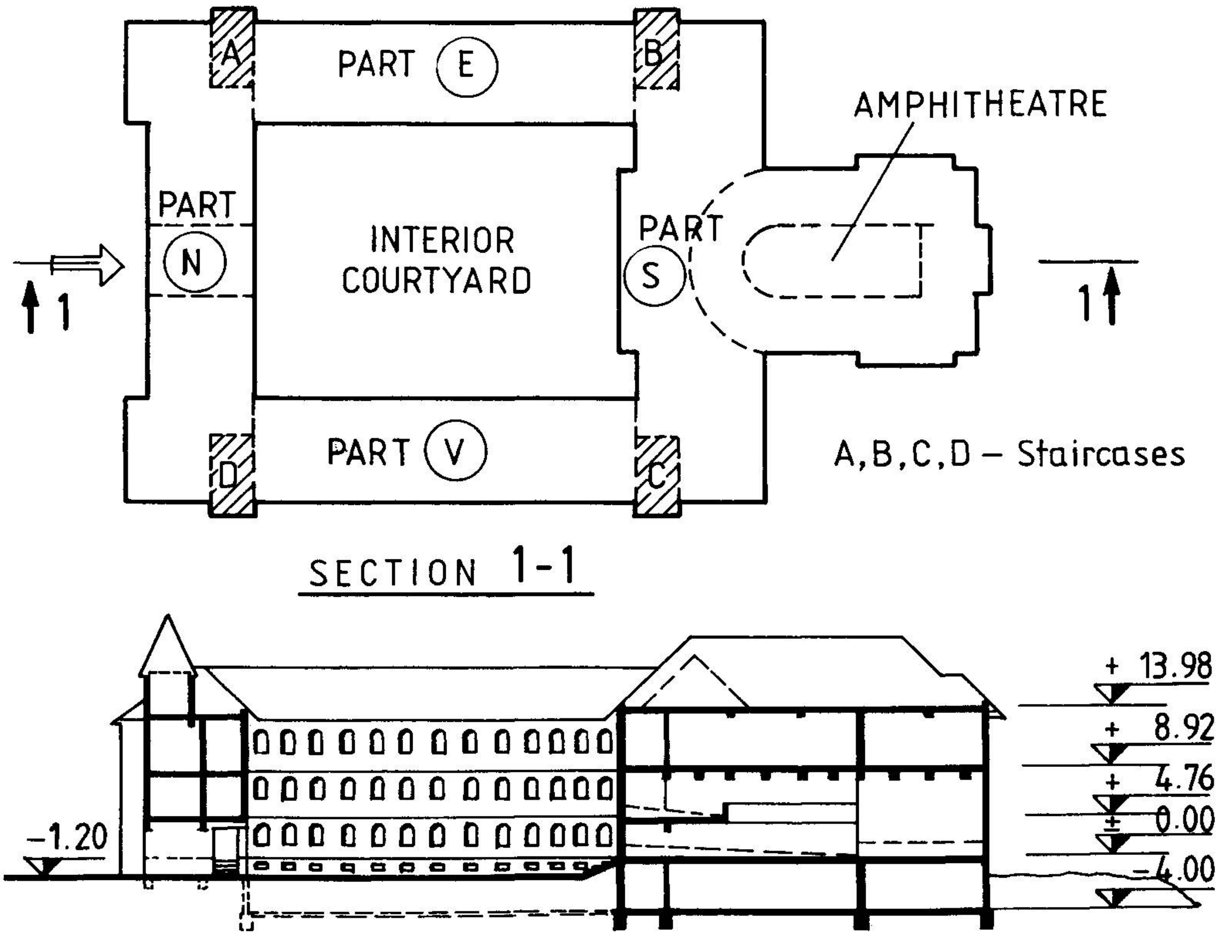

Fig. 1 The geometrical characteristics of the building

permanent threat for human lives [1].

One of these old buildings, damaged during the earthquake from March 4, 1977 with the epicentre in Vrancea - Romania, is the building of the Faculty of Agronomy from Craiova - Romania.

2. SHORT DESCRIPTION OF THE BUILDING

The building was erected in three stages, from 1928 to 1954, and its structure is made of resistance walls with a full brick masonry, placed after two principal horizontal directions, for the ground floor and the storeys, whereas the basement walls and the foundation are made of simple concrete.

Having a quadrilateral principal plane form, with an interior courtyard, and with an amphitheatre in the South part (see Figure 1), the building is developed on three levels (ground floor and two storeys) in the North part and four levels (basement, ground floor and two storeys) in the South, East and West parts.

The floors of the building are made of monolith reinforced concrete in all the wings and at all the storeys of the building, except the one that covers the amphitheatre from the South part of the building, which was replaced by a lath and plaster ceiling, suspended by the soles of the roof trusses.

The building has a timber roof with a covering of gutter tiles. For the vertical circulation, the building has four reinforced concrete stairs.

3. THE DAMAGES OF THE BUILDING DURING THE EARTHQUAKE

The main damages (as described in detail in [2]) were the followings:

- the cracking of the majority of the resistance walls (longitudinal and transversal walls), with the following characteristics: horizontal cracks under the floor girdles, vertical cracks at the appearance of the floor joints, crossed cracks from the shearing loads and cracks in the lintels of doors and windows (see Figure 2);

- the appearance of a number of joints in the floors, caused by the cracking of the floors in the connection area between the transversal and longitudinal wings of the building;

- the settlement of the South wing walls foundation and the cracking of these walls;

- the cracking of the walls and stair elements of the staircases "B", "C" and "A" (see Figure 1), with partial wresting of the stair landings from the exterior walls of the staircases "B" and "C";

- the fall of the chimneys, causing damages of the roof and of the altic floors;

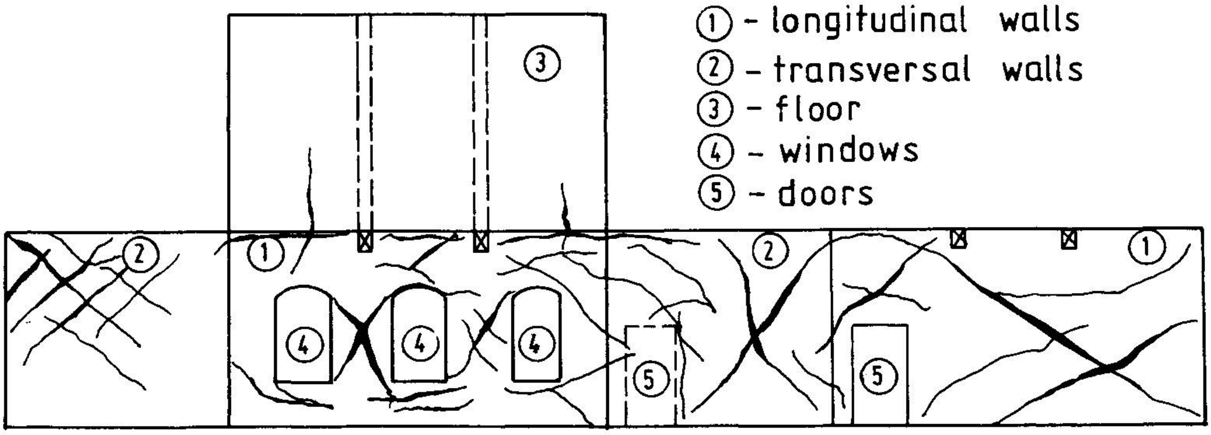

Fig. 2 An example of walls cracking

4. PROPOSALS FOR THE STRUCTURE CONSOLIDATION OF THE BUILDING

Because the masonry structure rigidity was diminished by the cracking and damaging of the walls and floors, the verification of the bearing capacity after the earthquake action was made according to the current Romanian norms: P2-85 [3] and P100-92 [4], with the following relation:

$$\eta S_0 \leq m \sum T_{cj,\min} \qquad (1)$$

where:

ηS_0 - represents the calculated load of the structure, under the effect of a violent earthquake (7.5 degrees on the Richter scale);

$m \sum T_{cj,\min}$ - represents the minimum bearing capacity, calculated with a simplifying assumptions acceptance [5].

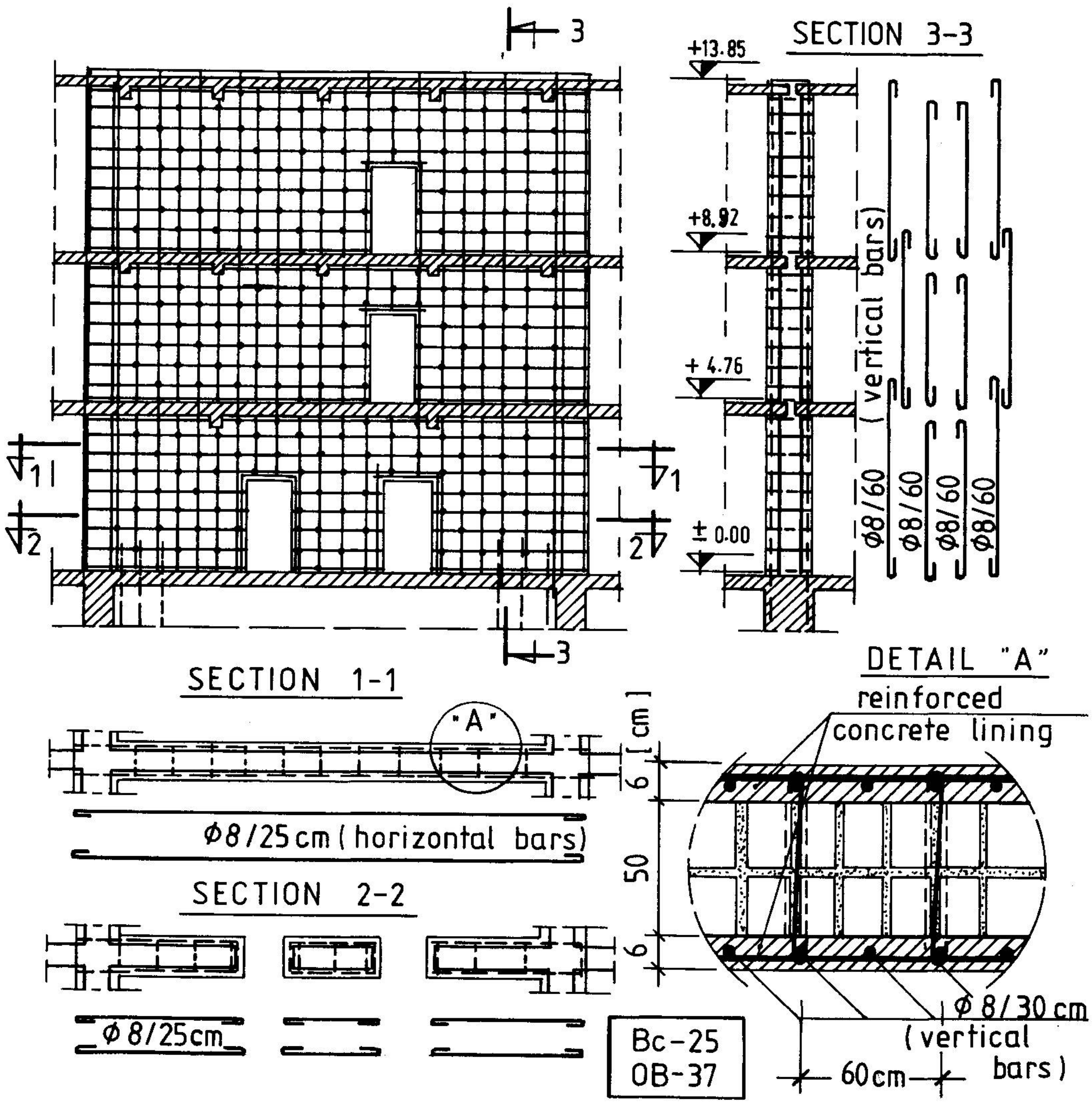

Fig. 3 The lining of the resistance masonry walls

The result of the calculus pointed out that masonry structure of the building does not comply with the antiseismic requests and that it cannot resist to a strong seismic load corresponding to the antiseismic protection degree in which Craiova is included.

The most important recommendations regarding the repairing and the consolidation of the structure were:

- the lining on both sides of the longitudinal and transversal walls of the structure, with thin monolith reinforced concrete shear walls (having a thickness of 6 cm). There must be pointed out the necessity of the continuity of the vertical reinforcements from the concrete shear walls on both sides of the masonry wall (by piercing the floors), and the linking of the steel reinforcements on both sides of the masonry wall, with reinforcement nets traversing the wall (see Figure 3).

- the making of eight reinforced concrete transversal frames in the South wing linked to the masonry walls and the existing floors (see Figure 4). The beams of these frames can be made including the existing monolith reinforced concrete beams of the floor, in the new beams of the frames and the monolith reinforced concrete columns can be made inside the two large rooms having their own foundations linked to the existing foundations of the walls.

- the consolidation of the amphitheatre by introducing the reinforced concrete transversal frames on the height of the basement, ground floor and first storey (see Figure 4). These frames will be linked with the exterior walls and with the existing transversal beams of the reinforced concrete floor above the amphitheatre;

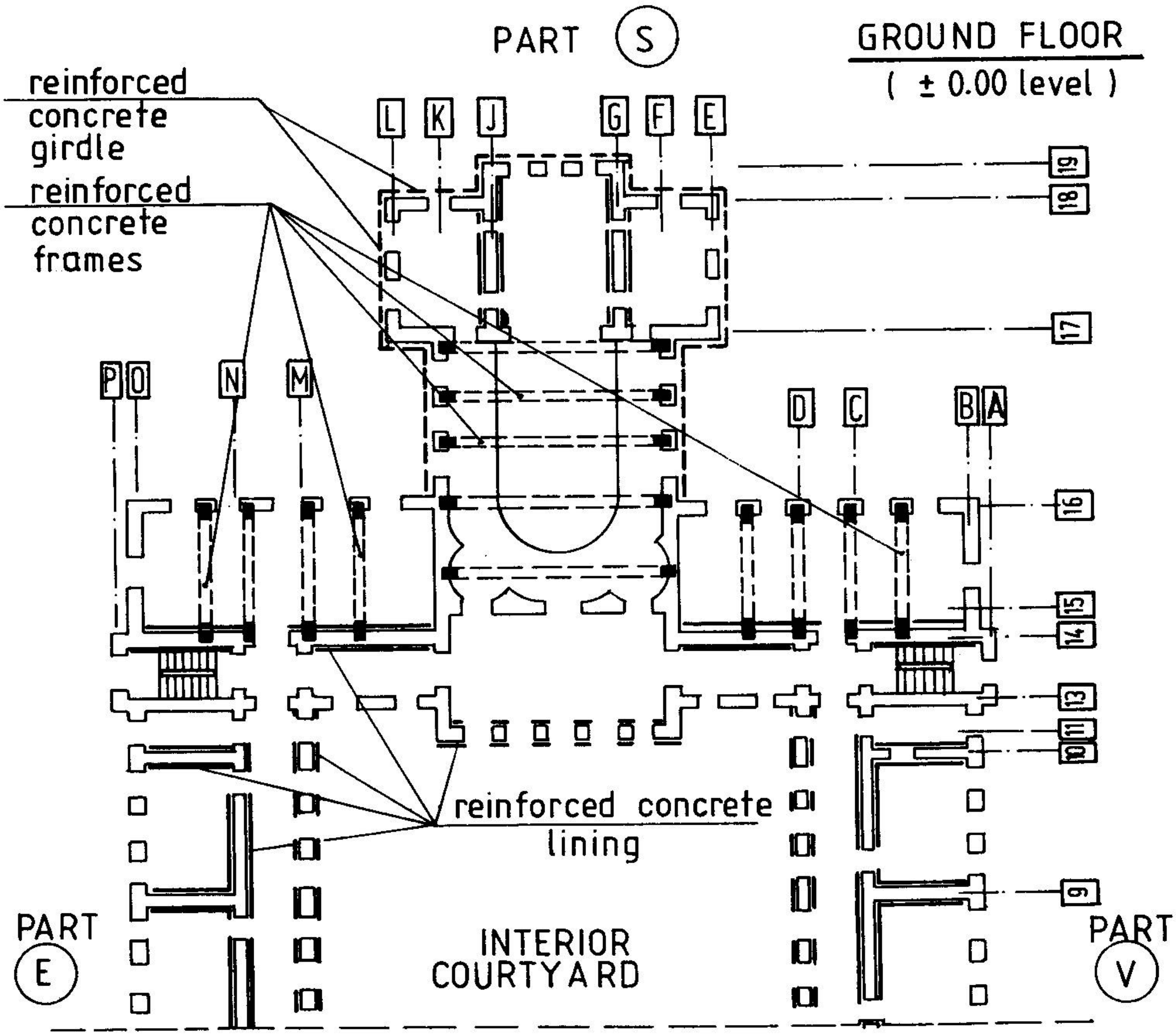

Fig. 4 The consolidation of the South wing of the building

- the repairing of the damaged staircases with partial masonry restorings, steel bars and girdles;

- the local repairing of the interior and exterior walls between windows and doors by lining with reinforcement nets and high resistance shotcrete;

- the making of some reinforced concrete girdles on the exterior of the amphitheatre, at the level of the floor situated above the basement, at the superior part of the windows railing and at the level of the floor above the amphitheatre. These girdles will be linked to the columns of the new frames by piercing the exterior walls.

- the consolidation of the exterior and interior corners of the North wing by lining the whole height of the building with reinforced concrete and by linking these corners with horizontal steel bars (tie rod) included in the exterior reinforced concrete girdles at the floors levels.

It can be concluded that the repairing and the consolidation based on a design that will take into consideration the above mentioned suggestions can contribute to the restoration of the structure rigidity and bearing capacity at least up to the level they were before the earthquake action.

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Evaluation and Analysis of a Masonry Structure for Seismic Loading

Evaluation et calcul d'une structure en maçonnerie sous charge sismique

Untersuchung und Berechnung eines Mauerwerkbaus unter Erdbebenbelastung

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SUMMARY

Several steps must be taken to restore or rehabilitate an existing structure for various imposed load conditions. A history and condition survey of an existing structure revealed an inadequate resistance to seismic forces. Field testing and inspection revealed deficiencies in the original construction which required conformance to current Building Code. A structural analysis was carried out to determine the structural adequacy of the masonry walls and to facilitate rehabilitation in an economical way. This paper deals with the evaluation of existing clay tile block masonry work for retrofitting the structure.

RÉSUMÉ

Pour divers cas de charges, il faut effectuer plusieurs démarches afin d'assurer la réparation et la consolidation d'une structure existante. L'étude d'un ouvrage a montré une résistance parasismique inadéquate. Les vérifications sur le site ont mis en évidence les défaillances dans la construction initiale, la rendant non conforme aux exigences actuelles des normes. Les auteurs ont déterminé à l'aide d'un calcul statique la résistance du mur en maçonnerie, ainsi que les possibilités économiques du renforcement à envisager.

ZUSAMMENFASSUNG

Um ein Tragwerk gegenüber unterschiedlichen Belastungszuständen wieder auf eine genügende Tragfähigkeit zu bringen, müssen mehrere Schritte unternommen werden. Am Beispiel eines Tragwerks mit mangelhaftem Erdbebenwiderstand wurde zuerst die Vorgeschichte und der gegenwärtige Zustand aufgenommen. Untersuchungen und Inspektionen enthüllten Mängel in der ursprünglichen Konstruktion und der Erfüllung der heutigen Normanforderungen. Mittels einer statischen Berechnung wurde die Tragfähigkeit der Mauerwerkswand und deren wirtschaftliche Sanierungsmöglichkeiten bestimmt.

INTRODUCTION

Rehabilitation of existing buildings has grown significantly in the construction industry during the current recession. Retrofitting of structures must comply with current building regulations and have structural adequacy to resist such imposed loading as earthquakes.

The building under study was initially constructed in 1969 with a hollow tile clay block construction.

Although structural clay tile was first produced in the United States of America in about 1875, archaeological excavations have proved that structures were built with clay burnt bricks as long as 5000 years ago. In 1921 ASTM proposed a standard for hollow clay tiles. Subsequently, the use of hollow clay tile block buildings was predominant between 1940 and 1960.

The building is a clay tile block, cavity wall single storey structure with structural steel open web steel joists supporting the metal deck roofing. The structure is located in Ottawa, Canada, which is a seismic zone. Since its construction, several changes have occurred in the Canadian Building Codes (in the last 20 years).

During the construction of an addition to the building in 1992, it was discovered that several cracks had developed at beam bearing locations at the load bearing walls of the original building. During the renovation, it was found that the existing clay tile blocks were not adequately reinforced or grouted and were defective in their original construction. Although there were no major visual deficiencies noted on the outside, it was decided to review the rest of the original building for its structural adequacy to resist gravity and seismic loads according to current code requirements.

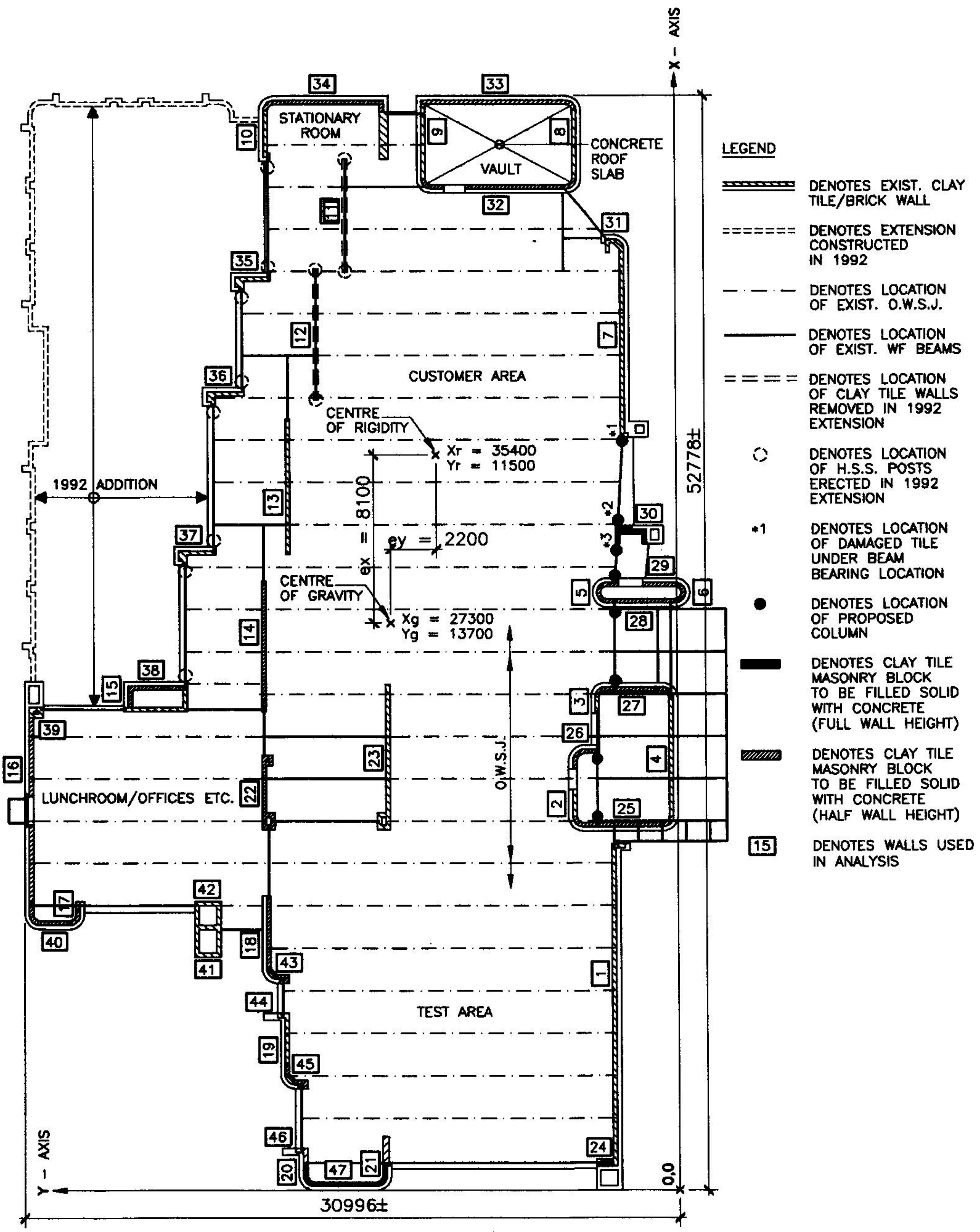
OBSERVATIONS

The layout of the building is shown on Figure 1. The walls are built with clay tile block and face clay brick as shown on Figure 3. Typical clay tile block used in the building is shown in Figure 2.

Inspection and investigation of the building revealed that there are several areas of deficient construction and inadequacies in the reinforcement of the original clay tile block masonry walls. The normal method to repair and restore the clay tile block walls would be to grout, reinforce, and restore the walls to the original design details conforming to current code requirements. The cost of such repairs would be substantial and the restoration would cause disruption to the operation of the building. Since the structure did not exhibit severe distress, it was decided to carry out a detailed structural analysis to establish the level of stresses in clay tile block masonry walls.

PURPOSE OF ANALYSIS

The purpose of this analysis was to determine if the unreinforced masonry walls were able to resist combined gravity and wind or earthquake loads in accordance with the Building Code. The current Building Code requires that load bearing and lateral load resisting masonry walls in velocity or acceleration related seismic zones of 2 and higher shall be reinforced. The Ottawa area is in an acceleration related zone of 4 and a velocity related zone of 2.



PLAN

Fig. 1 Building Layout

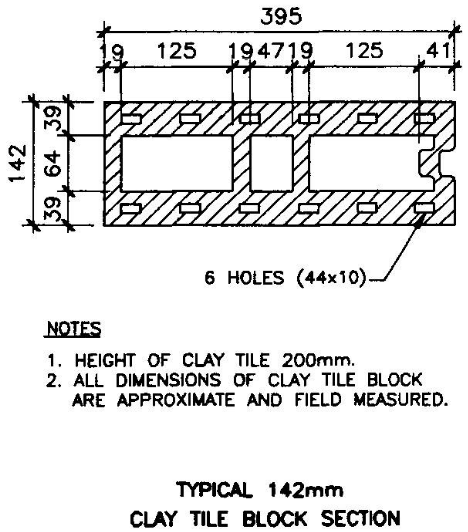


Fig. 2 Clay Tile Block

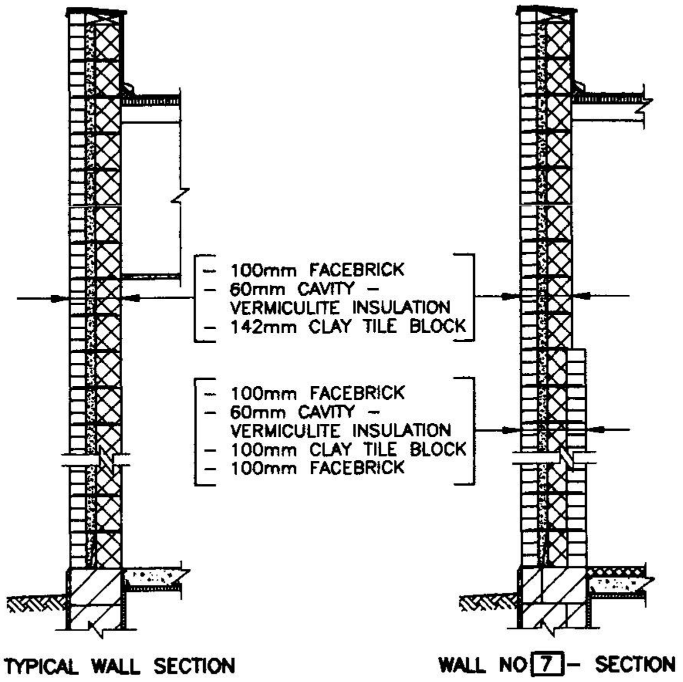


Fig. 3 Wall Sections

ANALYSIS MODEL

The wall system which resist gravity and earthquake loads is shown on Figure 1. The load resisting system consists of 45 individual walls. The exterior masonry cavity walls have been assumed to act integrally in resisting lateral forces. The clay tile block wall and brick wall are tied together with ladder type reinforcement. Gravity loads, however are supported by the loaded wythe of clay tile block masonry only.

The analysis is based on a relative wall stiffness method. Torsional effects due to horizontal forces are adequately dealt with by a stiffness matrix of the composite walls at various locations, as shown on Figure 1. Calculations indicated that seismic force controlled the analysis.

The horizontal component of earthquake load at the base of the structure V_e is determined from the following equation:

$$V_e = V \bullet S \bullet I \bullet F \bullet W \bullet \text{ as per O.B.C. 4.1.9.1 (5)} \quad [10]$$

Working stress design is considered in the evaluation of stresses to be compatible with the age of construction.

The total horizontal seismic forces along the building are distributed to walls based on their relative stiffness.

$$H_i = (K_i/\Sigma K) \bullet V \quad [8]$$

The design lateral earthquake force at the base of structure V = 1,190 kN. The design eccentricities are computed to obtain torsional moments in the orthogonal direction. The horizontal force in the walls (H_T) caused by the torsional moments (M_{tx}) is determined by the following equation:

$$H_T = [K \bullet d^2/\Sigma(K \bullet d^2)] \bullet M_{tx}/d \quad [8]$$

where d is the distance from the centre of gravity of the wall to the centre of rigidity of the structure.

The axial, flexural and shear stresses are computed. The critical stresses at various walls are shown in Table 1. The calculated stresses are compared with allowable stresses noted in Table 2 and the overstress at various wall locations are established.

WALL NO.	CALCULATED STRESSES					
	MAX. AXIAL STRESS (MPa)	MIN. AXIAL STRESS (MPa)	COMP. STRESS / ALLOW. STRESS	TENSILE STRESS / ALLOW. STRESS	SHEAR STRESS (MPa)	SHEAR STRESS / ALLOW. STRESS
14	0.311	−0.041	0.758	0.292	0.068	0.486
16	0.262	−0.076	0.640	0.541	0.117	0.834
18	0.343	−0.029	0.837	0.208	0.045	0.321
22	0.409	0.005	0.997	–	0.040	0.287
25	0.380	−0.105	0.927	0.751	0.131	0.938
26	0.311	−0.046	0.758	0.326	0.014	0.099
27	0.385	−0.068	0.940	0.484	0.102	0.729
28	0.374	−0.097	0.912	0.695	0.085	0.610
29	0.343	−0.094	0.837	0.672	0.084	0.599
30	0.452	0.065	1.103	–	0.012	0.084
32	0.308	−0.138	0.752	0.986	0.118	0.845
33	0.304	−0.140	0.741	1.000	0.119	0.850
34	0.295	−0.142	0.719	1.016	0.100	0.717
38	0.292	−0.137	0.713	0.977	0.047	0.332
40	0.351	−0.069	0.857	0.490	0.064	0.458
45	0.316	−0.089	0.770	0.635	0.016	0.114
47	0.438	−0.131	1.068	0.935	0.118	0.840

Notes:
Allowable Shear Stress = 0.14 MPa (15psi x 1.333 = 20psi, see Table 2 below)
Allowable Compressive Stress = 0.41 MPa (60psi)
Allowable Tensile Stress = 0.14 MPa (15psi x 1.333 = 20psi, see Table 2 below)
1 MPa = 145.04 psi

TABLE 1

ALLOWABLE STRESSES IN UNIT MASONRY					
CONSTRUCTION	ALLOWABLE COMPRESSIVE STRESSES (psi)			ALLOWABLE STRESSES IN SHEAR OR TENSION IN FLEXURE (psi)	
	MORTAR			MORTAR	
	TYPE M	TYPE S	TYPE N	TYPE M OR S	TYPE N
Cavity walls, solid and hollow units	70 (1)	60 (1)	55 (1)	15 (2)	10 (2)
	(1) On gross cross-sectional area of wall minus area of cavity between wythes. The allowable compressive stresses for cavity walls are based upon the assumption that the floor loads bear upon but one of the two wythes. When hollow walls are loaded concentrically, the allowable stresses may be increased by 25 per cent.			(2) Stresses may be increased one third, due to wind or earthquake either acting alone or when combined with vertical loads.	

Note: Information shown in Table 2 was obtained from 'Brick and Tile Engineering' by Harry C. Plummer.

TABLE 2

CONCLUSIONS

The analysis pinpointed areas of overstress due to earthquake loads. The results of the analysis coincided with the problem areas and overstressed locations in the field. It was also evident that the stress levels were not critical in several locations; hence those areas did not need to be reinforced to match the original structure as detailed.

Only areas overstressed would be repaired and reinforced to withstand code imposed loads by this analysis approach. The method followed by this approach of analysis would save a considerable amount of money (approximately C$200,000.00), downtime and inconvenience to the operation of the building and clients.

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