Friction dampers for seismic control of Concordia University library building

Avtar S.Pall
Pall Dynamics Ltd, Pierrefonds, Quebec, Canada

Vassily Verganelakis
Francis Boulva & Partners Ltd, Montreal, Quebec, Canada

Cedric Marsh
Centre for Building Studies, Concordia University, Montreal, Quebec, Canada

ABSTRACT: A novel structural system has been used in the design of a reinforced concrete building. By incorporating friction damped steel bracing in the rigid concrete frames, the earthquake resistance is dramatically increased. The system combines the inherent strength and stiffness of a braced frame with the energy dissipation capacity of the friction devices. Quasi-static and non-linear time history dynamic analysis show the superior performance of the device-equipped building relative to other building systems. During a major earthquake, a large portion of the seismic energy is dissipated by the devices with no dependence on ductility, so the main structural elements remain elastic without damage. Furthermore, the new system, while assuring added safety to the occupants and reduced damage to the contents, offers the benefit of significant savings in the initial cost of construction.

INTRODUCTION

During a major earthquake, a large amount of kinetic energy is fed into the structure. The manner in which this energy is consumed in the structure determines the level of damage. All building codes, including the National Building Code of Canada (NBC 1985), recognize that it is economically not feasible to reconcile the seismic energy within the elastic capacity of the materials. The code philosophy is to design structures to resist moderate earthquakes without significant damage, and to avoid collapse of the structure during a major earthquake. According to the NBC, the probability of a major earthquake is 10 percent in 50 years. During a major earthquake, the elastic buildings would be subjected to many times higher forces than the code specified quasi-static forces, but reliance for the survival of the building is placed on the ductility of structure to dissipate energy while undergoing inelastic deformations. This involves permanent damage causing bending, twisting and cracking of materials, and repair costs may be as economically significant as the collapse of the structure.

The problems created by dependence on the ductility of the structure, can be reduced if the seismic energy can be dissipated independently from the primary structure. Learning from the lessons of the Mexico City earthquake, the State of California passed an Assembly Resolution (ACR 55-Seismic Safety) in September 1985 that all new publically owned buildings, such as hospitals and educational institutions, must incorporate new seismic technology, and existing buildings retrofitted to increase earthquake resistance. Buildings designed with the current code of practice can not safeguard the building or its contents from damage. This resolution is based on the consideration that the past code philosophy was concerned with the survival of the building structure, but today there are new factors to be taken into account as buildings contain extremely sensitive and costly equipment, vital in education, business, and health, and records which are kept electronically must be protected. It is expected that future codes will be directed more towards the control of secondary damage.

This paper describes a novel structural system chosen for use in construction of the New Library Building Complex of the Concordia University, Montreal, Canada. By installing friction devices in steel cross bracings in the concrete frame building, a large amount of seismic

FIG. 1. VIEW FROM McKAY STREET

FIG. 2. VIEW FROM BISHOP STREET

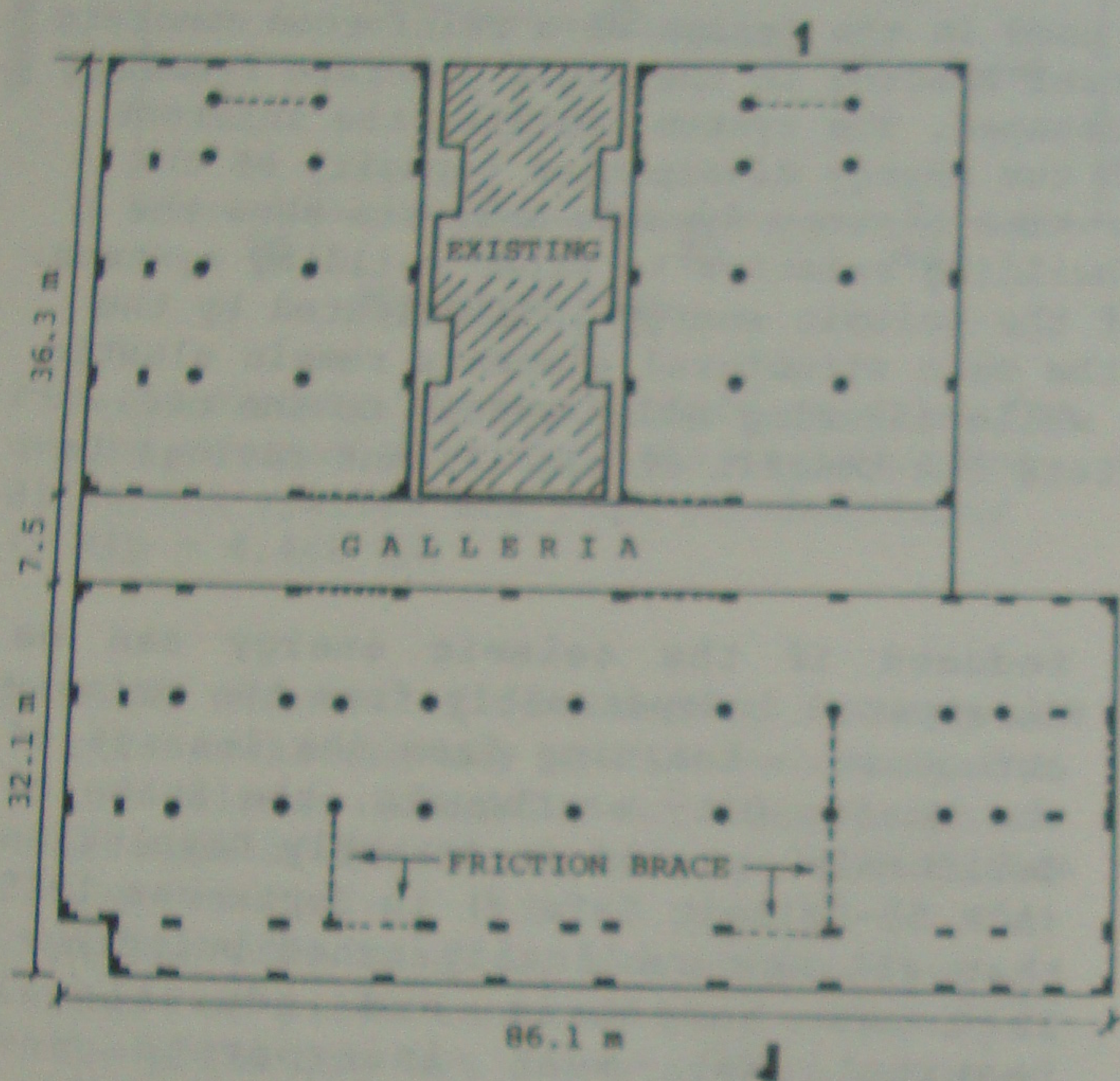


FIG. 3. GROUND FLOOR PLAN

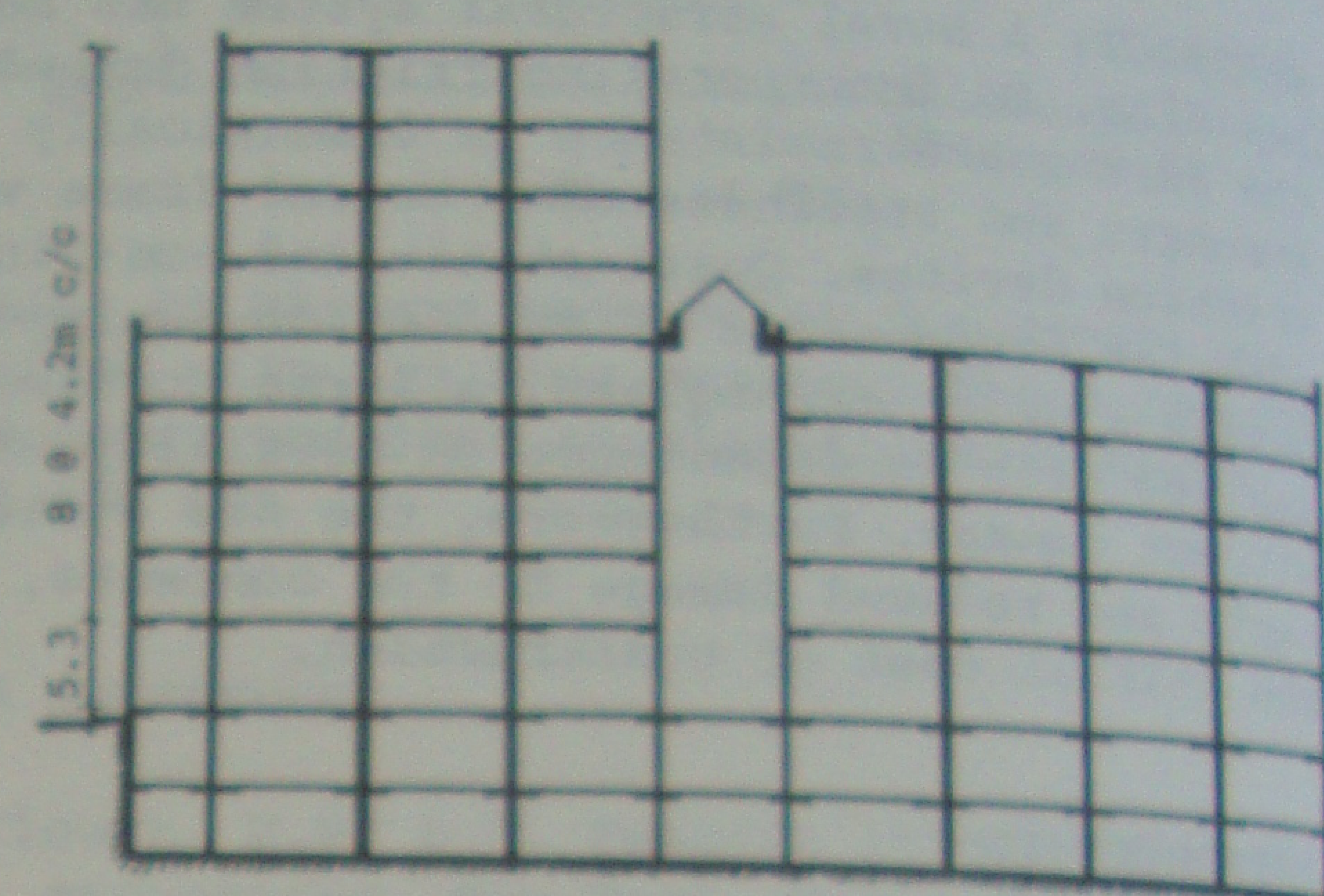
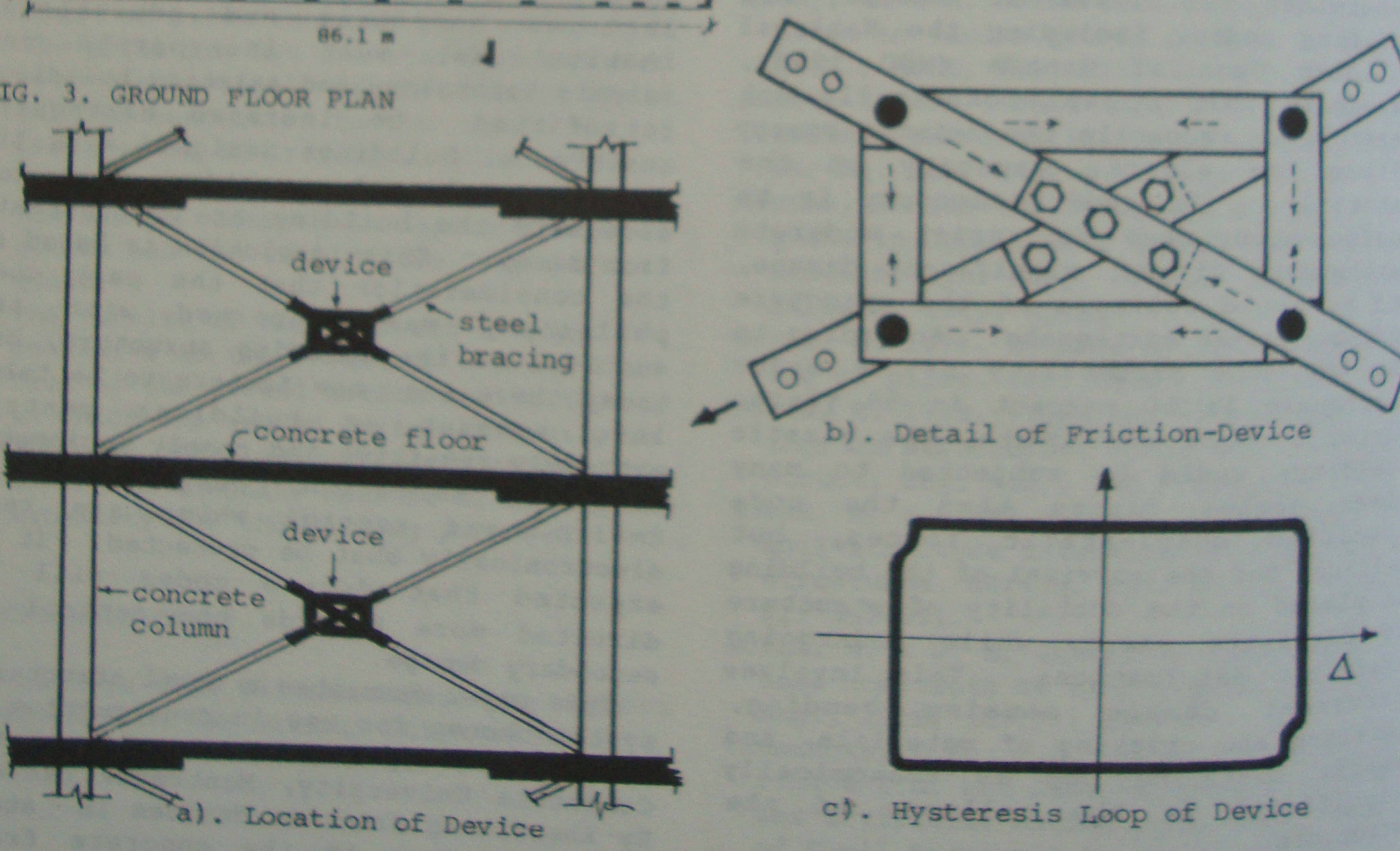


FIG. 4. CROSS-SECTION



a). Location of Device

b). Detail of Friction-Device

c). Hysteresis Loop of Device

FIG. 5. FRICTION DEVICE (DAMPER)

energy can be dissipated mechanically, while all the structural elements remain elastic. The friction devices act as safety valves to limit the forces exerted on the building and as structural dampers to limit the amplitude of vibrations. The concept provides a practical and economic approach to the problem of reducing the effects of earthquake forces.

DESCRIPTION OF THE BUILDING

The New Library Building Complex is located in downtown Montreal near the Hall Building of Concordia University. Figures 1 and 2 show the two views of the building. The complex consists of three new blocks, which enclose an existing 8 storey building on three sides. The plan of the building is shown in Figure 3.The main block is 86 m long, 32 m wide and 9 storeys above ground with 2 basements below ground. The total height of the structure above basement is about 50m. The other two blocks are each 36 m x 27 m and 5 storeys above ground with 2 basements below. The main columns are spaced at 9 m in both directions. These two blocks and the existing building are separated from the main block with a 7.5 m wide galleria of 5 storey height. Figure 4 shows the cross section of the building. The total covered area of the building is approximately 42,300 m^2. The lower 5 storeys of the building are to be used for the library. The upper 4 storeys are for offices. Parking and heavy storage areas are located in the basement.

Design Loading

The library floors are designed for a live load of 7.2 kN/m^2.

As the building is located in the centre of city with heavy concentration of tall buildings, it is sheltered from exposure to strong winds. The total wind load is only 0.5 kN/m^2.

With a probability of exceedance of 10% in 50 years,Montreal could have a major earthquake of peak horizontal ground accelerations of 0.18g (NBC Table J-2).

SELECTION OF STRUCTURAL SYSTEM

Fire hazard considerations for the library building and the availability of restricted clear storey height dictated the use of a reinforced concrete structure with flat slab construction.

State-of-the-Art

Ductile reinforced concrete frames are commonly used in earthquake regions. During major earthquakes, their performance has ranged from minor cracks to complete collapse depending on the way they were designed and detailed. In tall buildings, concrete shearwalls are incorporated to provide lateral rigidity. Generally, such structures are subjected to higher inertial forces during earthquakes and thereby place higher demand on strength and ductility. The ductility in a reinforced concrete wall is extremely sensitive to detailing and quality control. The presence of construction joints and lapping of vertical reinforcement, all at floor level, are typical of the problems which often cause the ductility of shearwalls to be viewed with suspicion (Allen 1973). This is true even though recent tests (Cardenas 1973, Bertero 1977) have shown that properly detailed slender concrete shearwalls do possess sufficient ductility. In any event, it is desirable that less dependence be placed on ductility, and that other means be found to dissipate seismic energy.

Coupled shearwalls with improved reinforcement detailing in coupling beams (Paulay 1977) have been suggested so that they can endure longer the damaging effect of inelastic energy dissipation, and delay yielding in the main structural walls. Even in this case the coupling beams are sacrificed to protect the shearwalls and will require major repairs. Slits in the concrete infill panels for framed buildings have been proposed (Muto 1973). The artificial joints create planes of weakness and thereby increase the ductile qualities under shear distortions. In order to reduce problems created by dependence on ductility of the structure, friction -damped concrete shearwalls have been suggested (Pall,Marsh 1981). In this, the seismic energy is dissipated mechanically by the friction joints as the walls deform.

In the past few years, the concept of steel bracing has been extended to provide lateral stiffness to concrete frames. This seems to be an economic alternative to the construction of expensive shearwalls. Single storey 1/3 scale RC frames with steel bracings have been tested in Japan (Shimazu, Fakuhara

1980). They concluded that the lateral strength of the frame can be increased up to about four times. Reinforced concrete frames damaged during earthquakes have been strengthened using steel bracing (Sugano, Fujimura 1980 and Vallee 1980). In another study (Kapur and Jain 1983), the elastic response of shearwall frames and braced concrete frames was evaluated. They concluded that the braced concrete frame is better than the shearwall frame in both response and cost. Inelastic seismic response of RC frames with k-bracing and x-bracing has been studied by Jain (1985), who concluded that the response of the frame with x-bracing is better as it places less demands on ductility.

The inelastic seismic response of friction-damped steel frames has been studied. The response of a 10 storey friction-damped braced frame, equipped with the friction devices, was compared to the response of a conventional moment resisting frame and a braced moment resisting frame (Pall,Marsh 1982 and Pall 1984). It was shown that the response of the friction-damped braced frame is much superior when compared to the other framing systems. Researchers at the University of California at Berkeley (Austin, Pister 1985) made independent studies of friction-damped braced frames, and concluded that they offer the benefits of savings in material costs and reduce damage.

Recently, large scale 3 storey model frames were tested on a shaking table at the University of British Columbia at Vancouver (Filiatrault, Cherry 1986). Tests were carried out on three types of frame having the same sectional properties. These were a moment resisting frame, a braced moment resisting frame and a friction-damped braced frame. In the case of braced frame, following the first few shocks, which caused the braces to yield, the response approached that of a moment resisting frame. The response of the friction-damped braced frame was much superior to that of the moment resisting frame and of the braced moment resisting frame. Even an earthquake record with a peak ground acceleration of 0.9g did not cause any damage to the friction-damped braced frame, while the other two frames suffered large permanent deformations. These tests confirmed the results of the analytical studies and demonstrated the operability of the new system.

Friction-Damped Braced Concrete Frame

In the chosen structural system, each steel cross bracing in the concrete frame is provided with a friction device. The device is designed not to slip under normal service loads, wind storms or moderate earthquakes. During a major earthquake, the devices slip at a predetermined load, before yielding occurs in the other structural elements of the frame.Slippage in the device then provides a mechanism for the dissipation of energy. As the braces carry a constant load while slipping, the additional loads are carried by the moment resisting frame. In this manner, redistribution of forces takes place between successive storeys, forcing all the braces to slip and participate in the process of energy dissipation. Such a modified structure combines the following characteristics:

1. It behaves like a braced frame structure during service load conditions, wind storms or moderate earthquakes and possesses sufficient stiffness to control lateral deflections.
2. During a major earthquake, a large portion of the seismic energy is dissipated mechanically in friction, thereby avoiding, or at least delaying, the yielding of main structural elements.
3. The natural period of the building varies with the amplitude of the oscillations, i.e. severtiy of the earthquake. Hence, the phenomenon of resonance or quasi-resonance is avoided.

FRICTION DEVICES

Friction devices have been developed with different configurations to suit particular construction needs. The general mechanism of a typical device is shown in Figure 5. When the tension in either of the diagonal braces forces the device to slip, it activates the mechanism to cause the short compression link within the device to slip simultaneously, even though the force in the compression diagonal outside the device may be zero. The compression diagonal is thereby shortened by the same amount as the tension diagonal is lengthened. In this manner, energy is dissipated in each half cycle. The surfaces in contact with the friction pads are of stainless steel to provide reliable slip load over the life of the building.

Cyclic dynamic tests have been conducted on the typical devices (Pall,Marsh,Fazio 1980 and Filiatrault, Cherry 1986). The performance is reliable and repeatable. The hystereses loops are rectangular with negligible fade over many cycles of reversals that can be encountered in successive earthquakes. A typical hysteresis loop is shown in Figure 5c. The devices offer reliable, inexpensive and maintenance free performance throughout the life of the building. They are always ready to do their job regardless of how many times they have performed in earthquakes.

Patented friction devices are designed and supplied by Pall Dynamics Limited.

Optimum Slip Load

The seismic response of a structure is determined by the amount of energy fed-in and energy dissipated. The optimum seismic response, therefore, consists of minimizing the difference between the input energy and energy dissipated.

The input energy is basically dependent on the natural period of the structure and the dynamic characteristics of the ground motion. The resonance can be avoided by modifying the dynamic characteristics of the structure relative to the forcing motion, however, ground motion characteristics are highly erratic. In a device-equipped building, the period of the structure is influenced by the slip load of the joint and varies with the amplitude of the oscillations, thus resonance is difficult to establish.

The energy dissipation is proportional to the product of slip load and the slip travel during each excursion. For very high slip loads, the energy dissipation in friction will be zero, as there will be no slippage. If the slip load is very low, the amount of energy dissipation again will be negligible. Between these extremes, there is an intermediate value to give the maximum energy dissipation.

By the proper selection of the slip load, it is possible to "tune" the response of the structure to an optimum value. Parametric dynamic studies have shown that the optimum slip load is independent of the time history of the earthquake motion and is rather a structural property. Also, within a variation of ± 20% of slip load, the seismic response is not significantly affected.

DESIGN PROCEDURE

The existing prescriptive formulas in the load definition chapter of the National Building Code of Canada are inappropriate for use in buildings using new systems. The use of devices offers the advantage of designing the structure with reduced level of earthquake load for the same degree of seismic protection, but until the codes are amended, the design procedure outlined below is followed in order to avoid delay in approval of plans and to meet the professional liability requirements.

1. Design the building frames without bracing for dead, live and wind loads to the requirements of the NBC.

2. Introduce the bracings into the moment resisting frames. Analyse and check the modified structure for code specified quasi-static lateral earthquake forces. Since the devices are designed not to slip at quasi-static forces, the presence of the device in the bracing is not considered at this stage. The members already designed for dead, live and wind loads will be checked for additonal stresses due to earthquake loads.

The braced frame as designed above, will meet all the statutory requirements of the NBC and the legal obligations of the structural engineers.

3. Non-linear time-history dynamic analysis is then carried out to assess the seismic response of the device-equipped frame during major earthquakes.

SEISMIC ANALYSIS

To demonstrate the influence of friction devices on the seismic response and to compare the results with conventional structural systems, a family of three types of frame, as shown in Fig. 6, were chosen for analysis. They were: moment-resisting frame (MRF), shearwall frame (SWF) and friction-damped frame (FDF). These were analysed for both quasi-static earthquake loads and non-linear time-history dynamic analysis. The findings for a typical 9 storey frame are discussed below.

Static Analysis

The three frames were subjected to lateral wind and quasi-static earthquake forces to the requirements of the NBC. Uncracked section of the members was

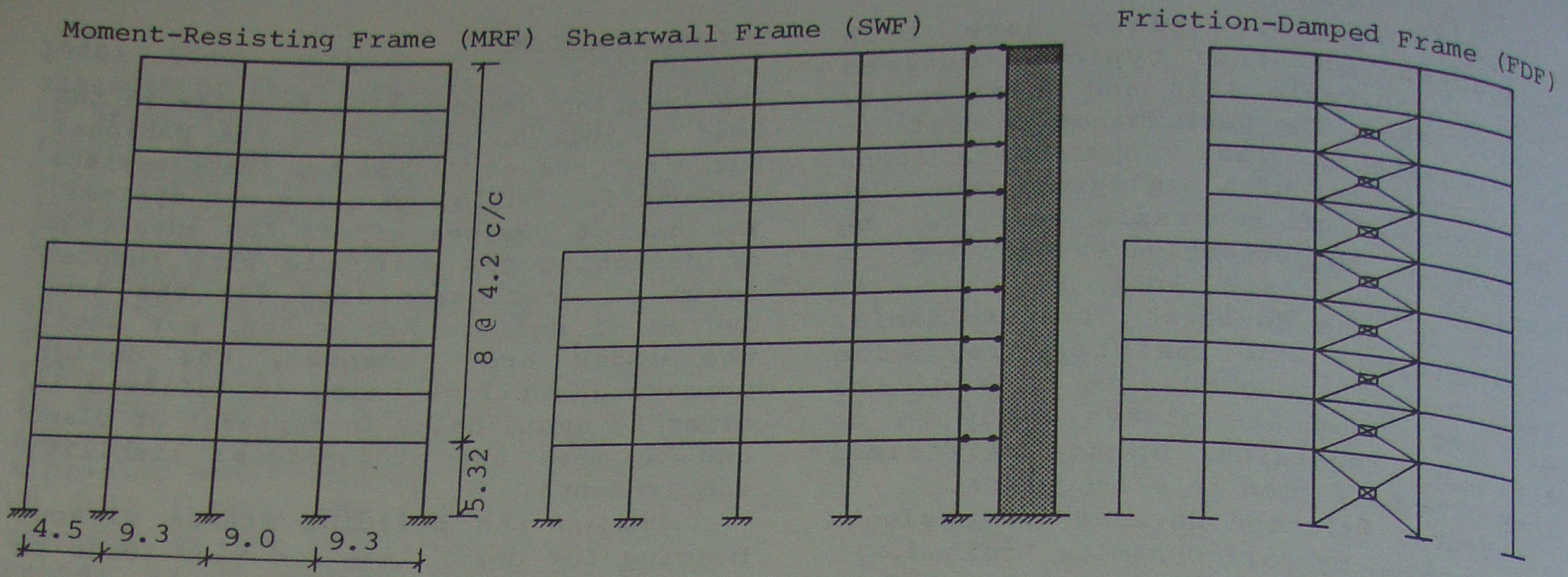


FIGURE 6. FRAMES STUDIED

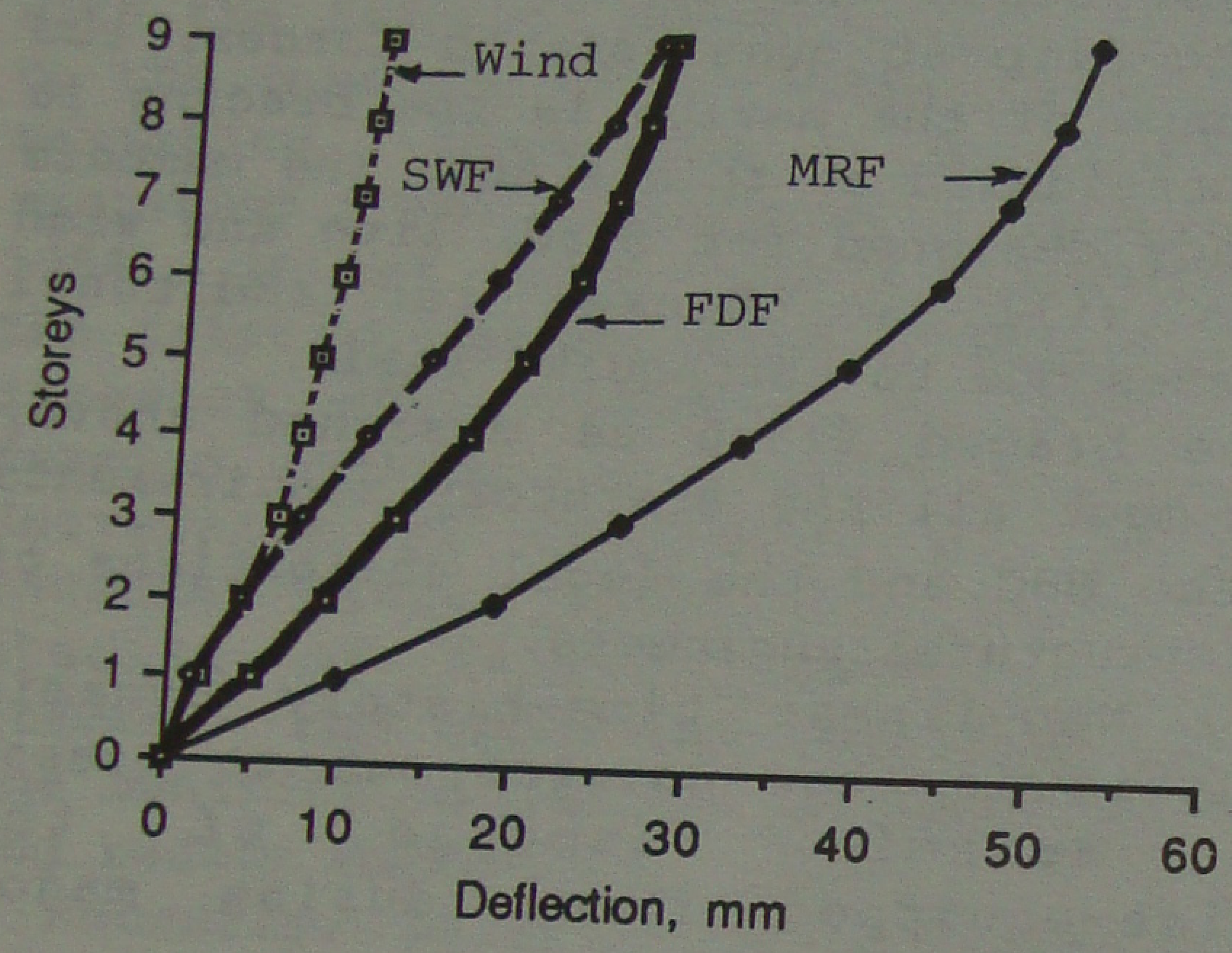


FIG. 7. BUILDING DEFLECTION (Static)

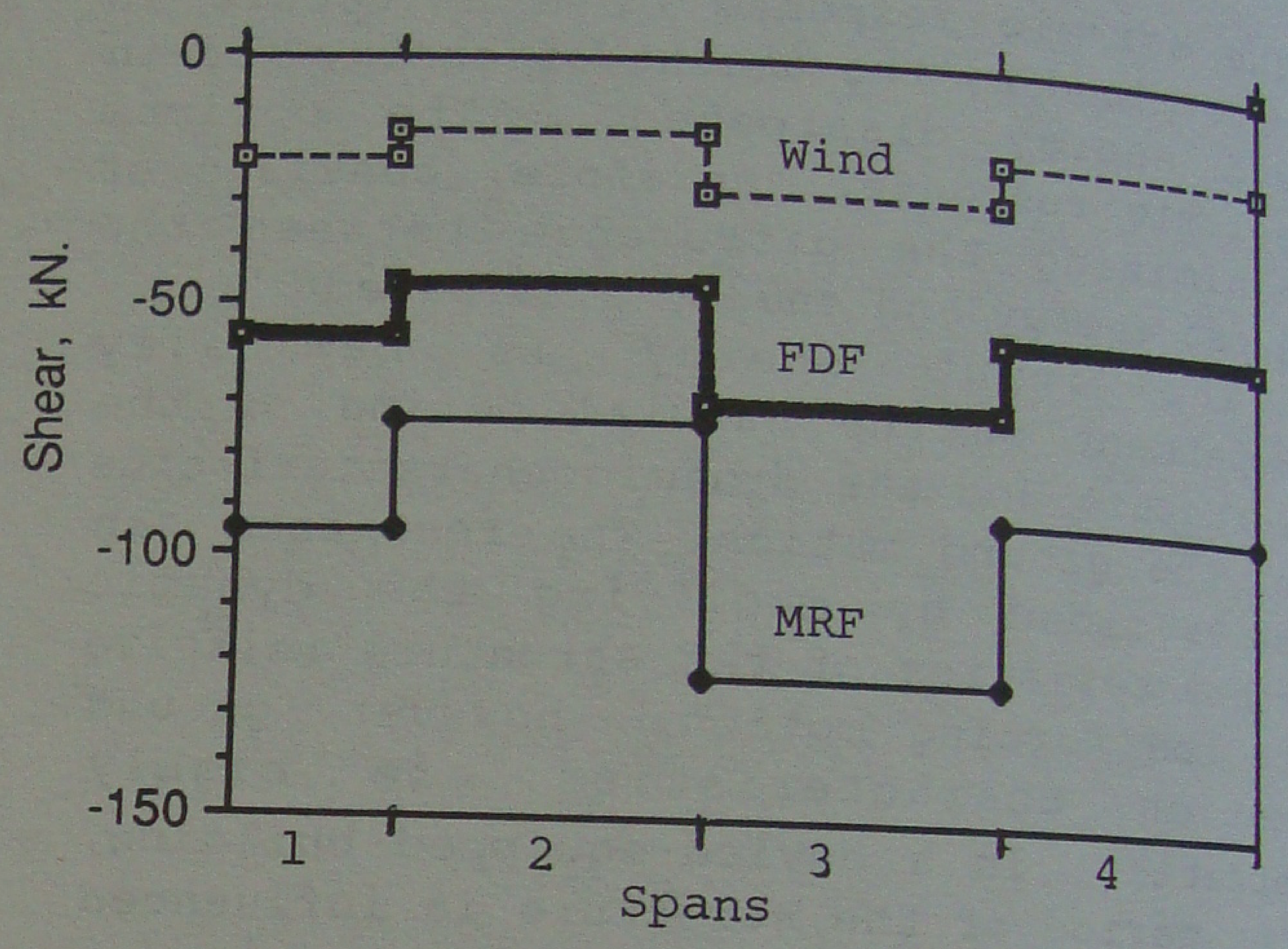


FIG. 8. SLAB SHEARS (Static)

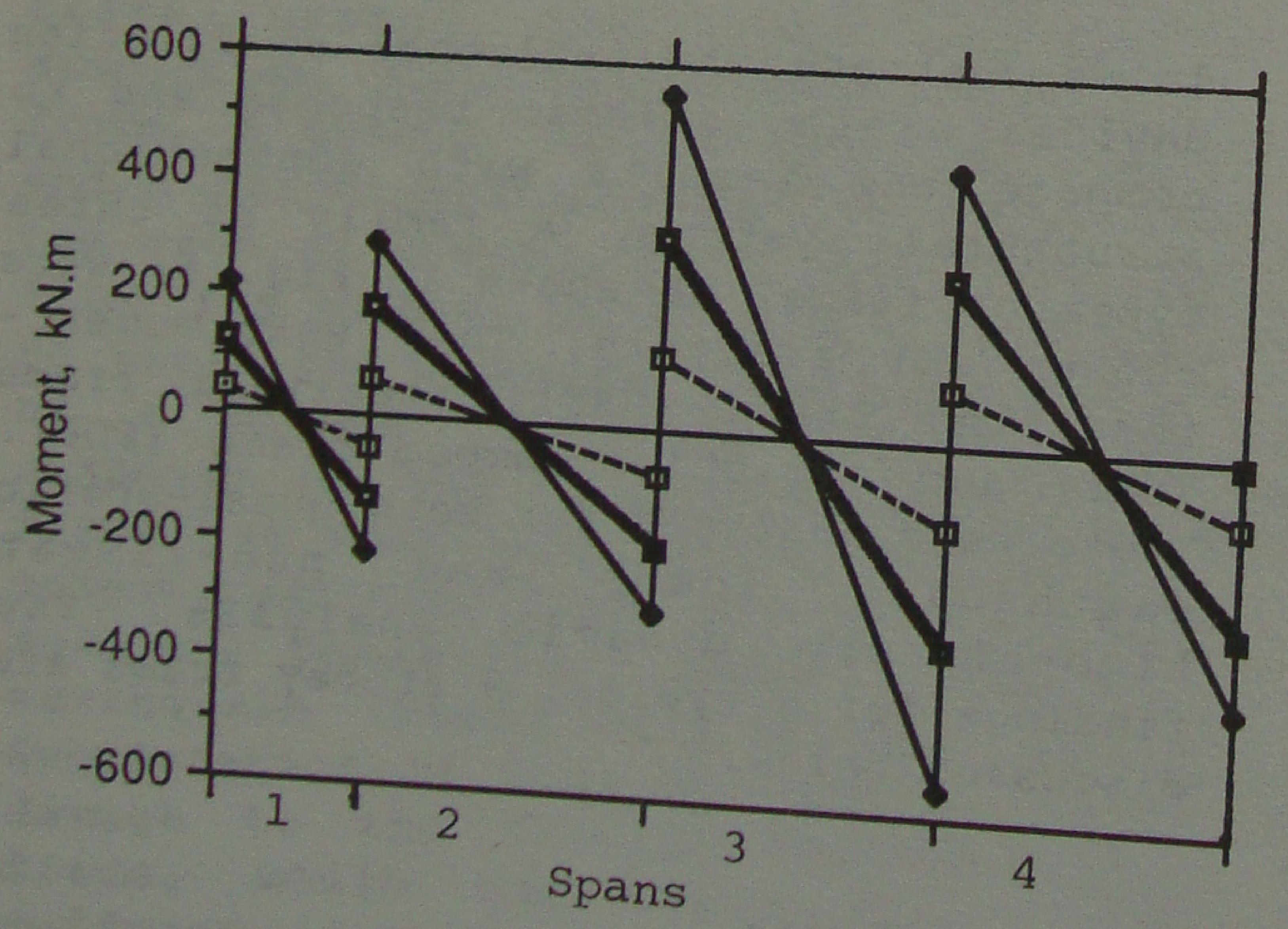


FIG. 9. SLAB MOMENTS (Static)

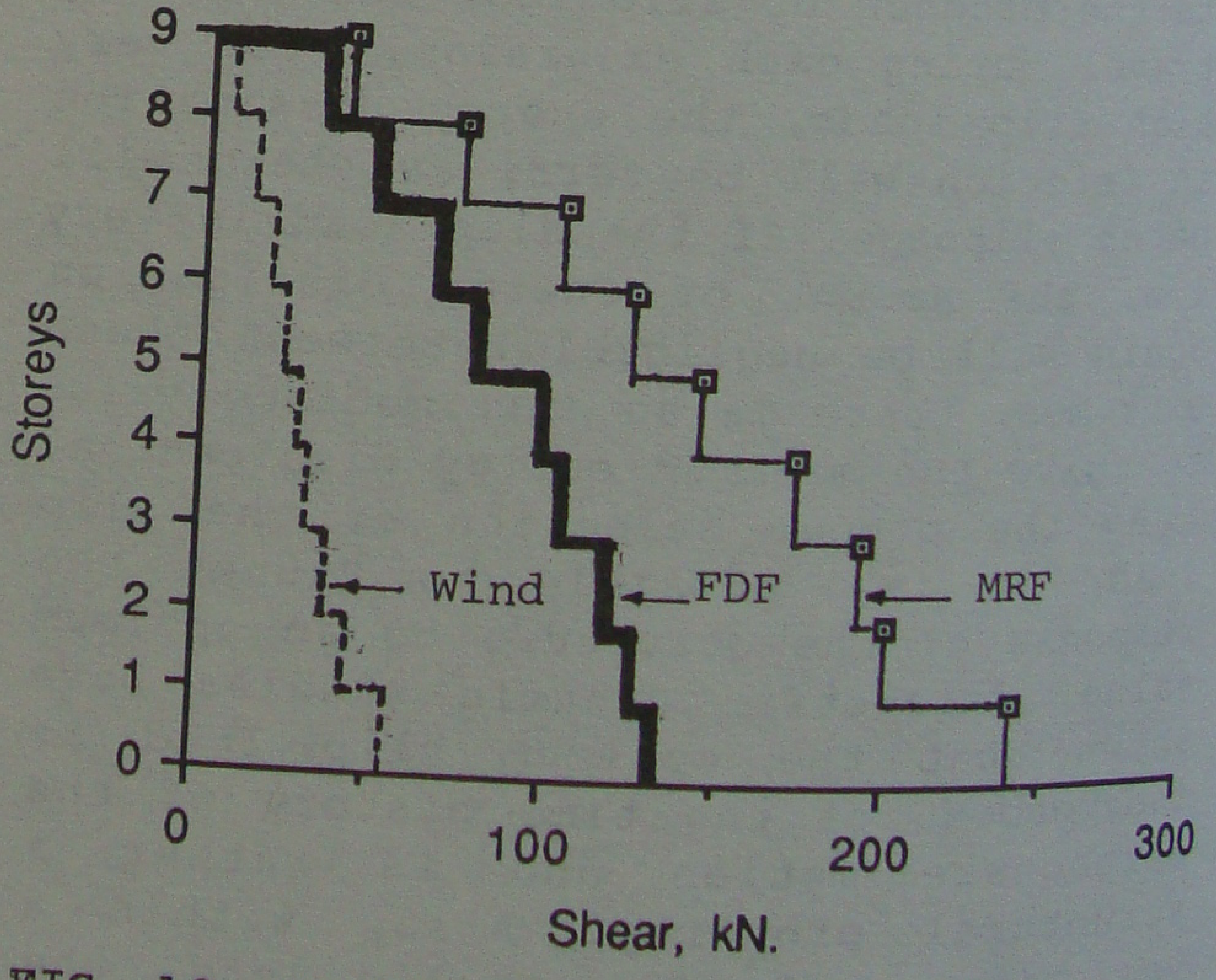


FIG. 10. COLUMN MOMENTS (Static)

assumed in the elastic analysis. The results of the analysis are shown in Figures 7 to 11.

1.Due to wind, the building deflection, shear and moments in both columns and slabs, are only 20-30% of those due to quasi-static earthquake forces. Hence the earthquake loading governs the overall design.

2.For earthquake loads, the deflections, shears and moments in both columns and slabs of the FDF are only 40-50% of the MRF.

3.In the SWF, the deflections at the lower storey are less than the FDF but almost converge to the same deflection at the top.

4.The behaviour of the SWF is typical of the shearwall frame interaction. While the shearwall supports the frames at the lower storeys, the frame supports the shearwall at the upper storeys. Conversely, the frames at the upper storeys are getting pulled by the cantilever action of the shear wall.

5.The behaviour of the SWF and the FDF are nearly identical for quasi-static forces.

6.The base shear due to quasi-static earthquake loads, is equivalent to 0.027g times the total mass of the structure.

Non-Linear Time-History Dynamic Analysis

According to the NBC, Montreal could have a major earthquake of peak horizontal ground accelerations of 0.18g. Non-linear time-history dynamic analysis was carried out using time histories of an artificial earthquake generated to match the response spectrum of Newmark-Blume-Kapur, which forms the basis of the NBC response spectrum. The peak ground accelerations of this earthquake were scaled to 0.18g. The computer program"DRAIN-2D", developed at the University of California, Berkeley, was used for this analysis.

Viscous damping of 5% of critical was assumed in the initial elastic stage to account for the presence of non-structural elements. Hysteretic damping due to inelastic action of structural elements and slipping of friction devices is automatically taken into account by the program. Reduction in initial stiffness of 25% for columns and 50% for slabs was assumed in all the frames. The Takeda model for the degrading stiffness of flexural concrete members was used. The integration time step was 0.005 seconds. Flexural and axial deformations were considered. Interaction between axial force and moments for columns and P-Δ effect were taken into account by including the geometric stiffness based on the axial force under static loads. Foundations of the structure rest on rock, hence soil structure interaction was neglected.

A series of analyses were made to determine the slip load to get the optimum response. The optimum slip load varies from 500-700 kN per storey depending upon the location of the device. However, there was little variation in response for ±20% of the optimum slip load. A total of 60 devices was required in this building to provide the desired energy dissipation.

The effectiveness of the friction device in improving the seismic response is seen in comparison of the results with the MRF and the SWF. The results of the non-linear time history analysis are shown in Figures 12 to 17 and are discussed below:

1.Time histories of deflection at the top of the MRF is 276 mm, which is about H/140, and exceed the code specified value of H/200. For these storey drifts, it is expected that the architectural finishes and fixtures will be badly damaged. In the case of the FDF the peak amplitude is 40% less than the MRF and the storey drifts are within the specified limit. The peak deflection of the SWF is close to that of the FDF, and follows its time history path. After the earthquake, both the FDF and the SWF nearly return to the original alignment whereas there is a permanent deformation of 26mm in the MRF.

2.Column shears and column moments for an interior column are shown in Figures 13 and 14. It is seen that in the case of MRF, the columns have yielded in 5 storeys. The forces in the FDF are about 60-70% of the MRF and all columns are elastic.

3.Moments and shears of slabs in an interior bay are shown in Figures 15 and 16. Again in the MRF, slabs yielded at 5 floors whereas all the slabs of the FDF are elastic.

4.Damage experienced by the three frames is shown in Figure 17. It is seen that in the case of the MRF, 50% columns and 33% of the slabs have yielded. In the SWF, the shear wall itself yielded at the base and two upper levels, 2 columns and 9 slabs yielded. All members of the FDF remained elastic without damage. In effect, the FDF could endure a much stronger earthquake.

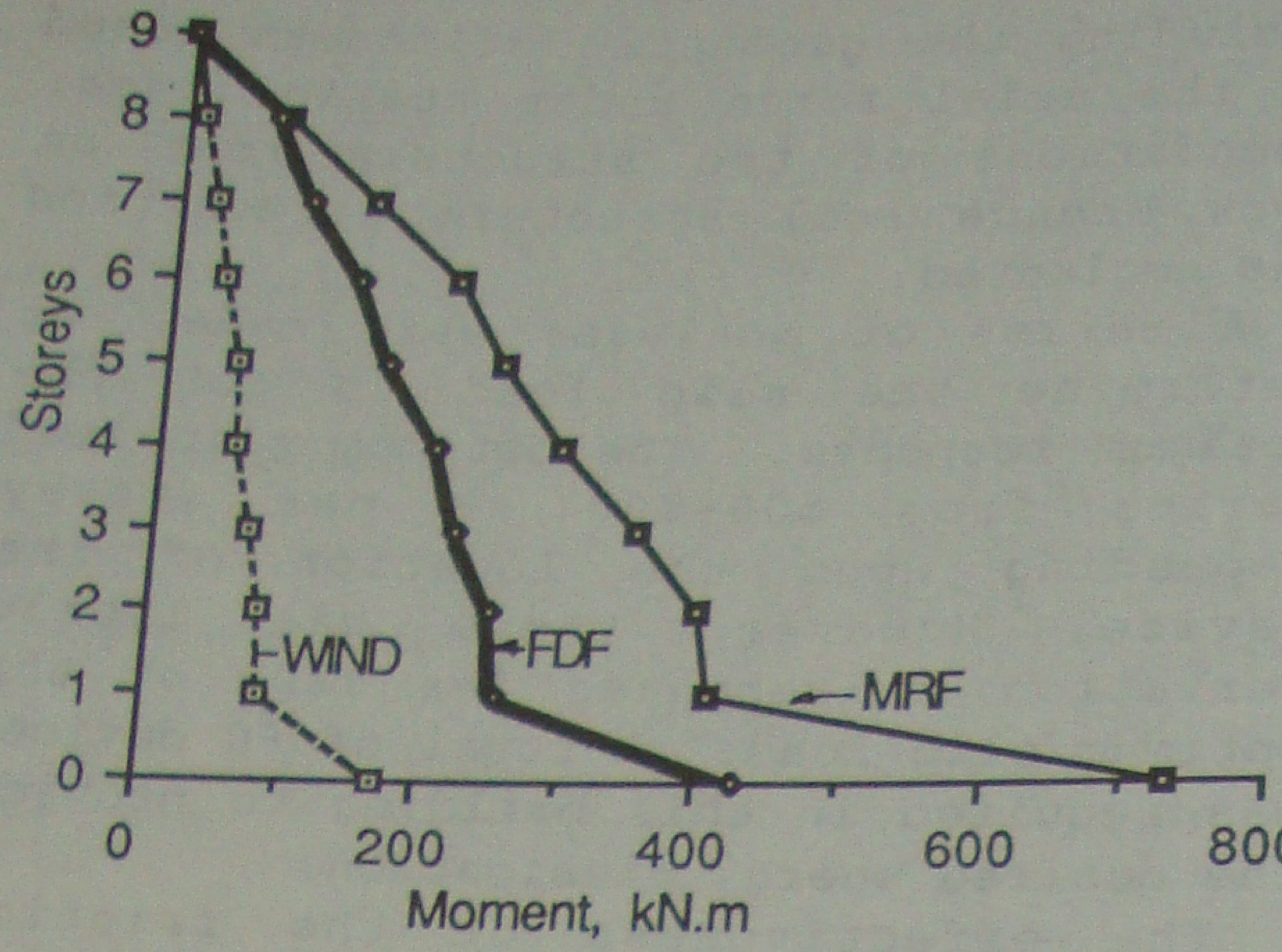


FIG. 11. COLUMN MOMENTS (Static)

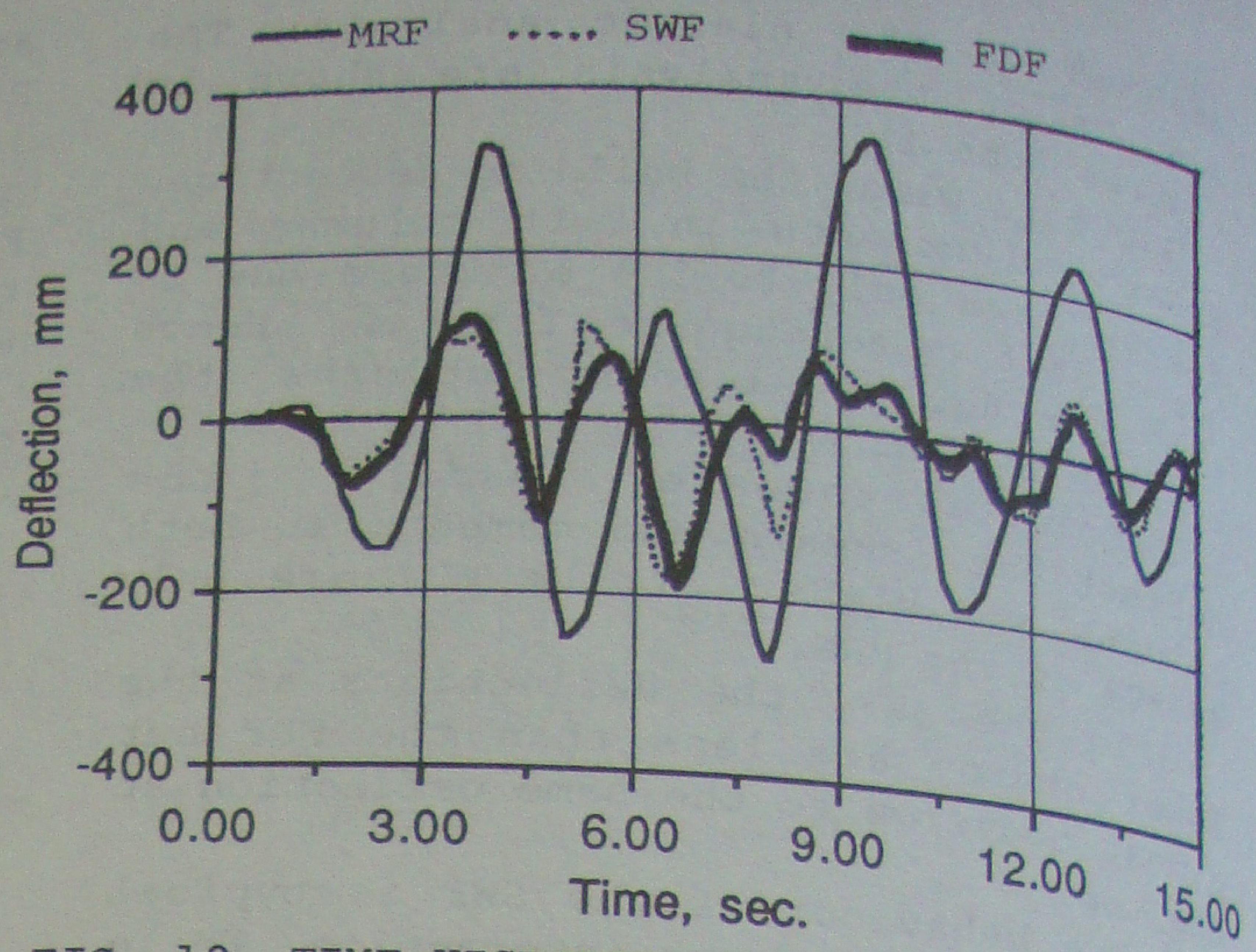


FIG. 12. TIME-HISTORIES OF DEFLECTION AT TOP (Nonlinear)

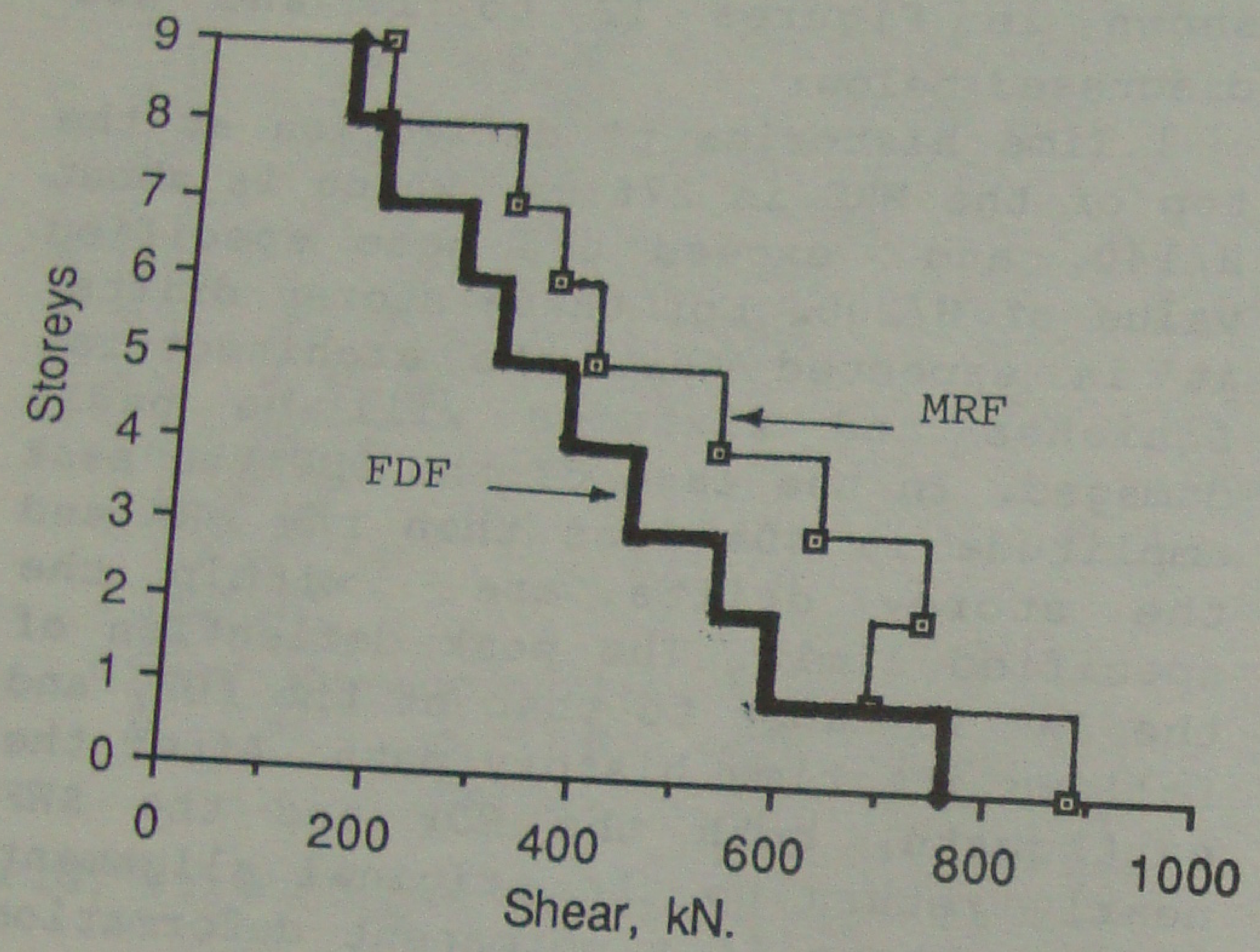


FIG. 13. SHEAR ENVELOPE, COLUMNS (Nonliear)

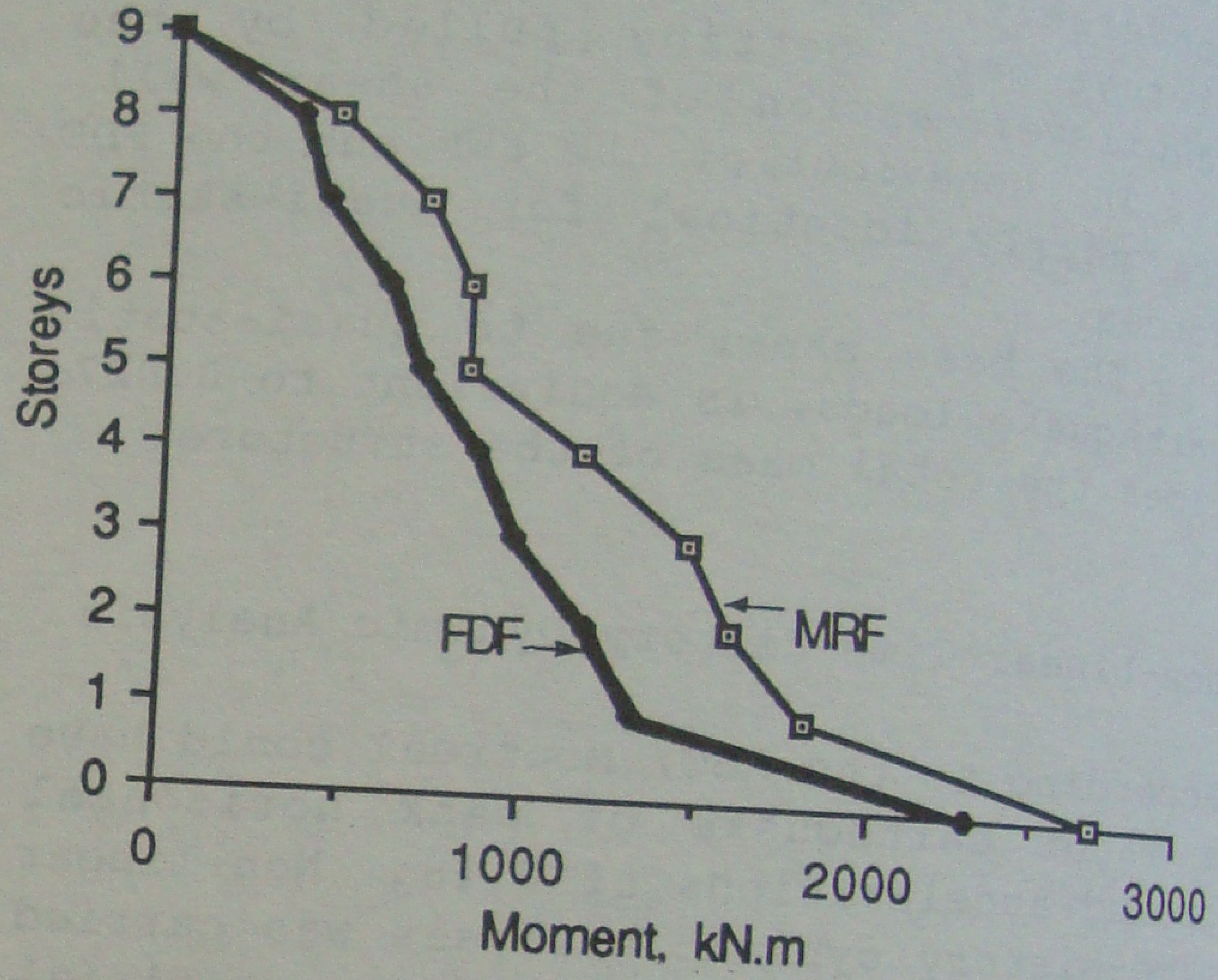


FIG. 14. MOMENT ENVELOPE, COLUMNS (Nonlinear)

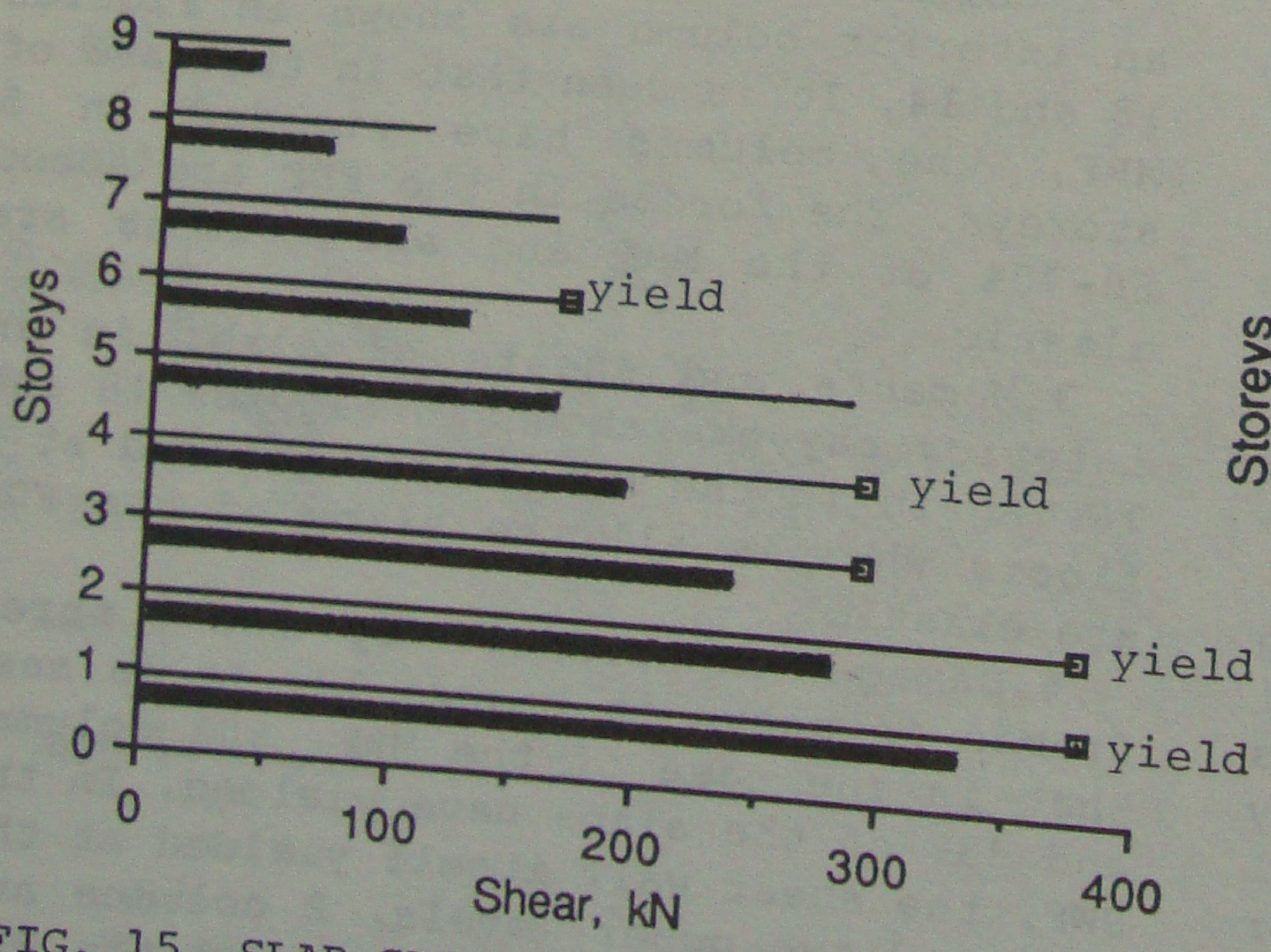


FIG. 15. SLAB SHEARS (Nonlinear)

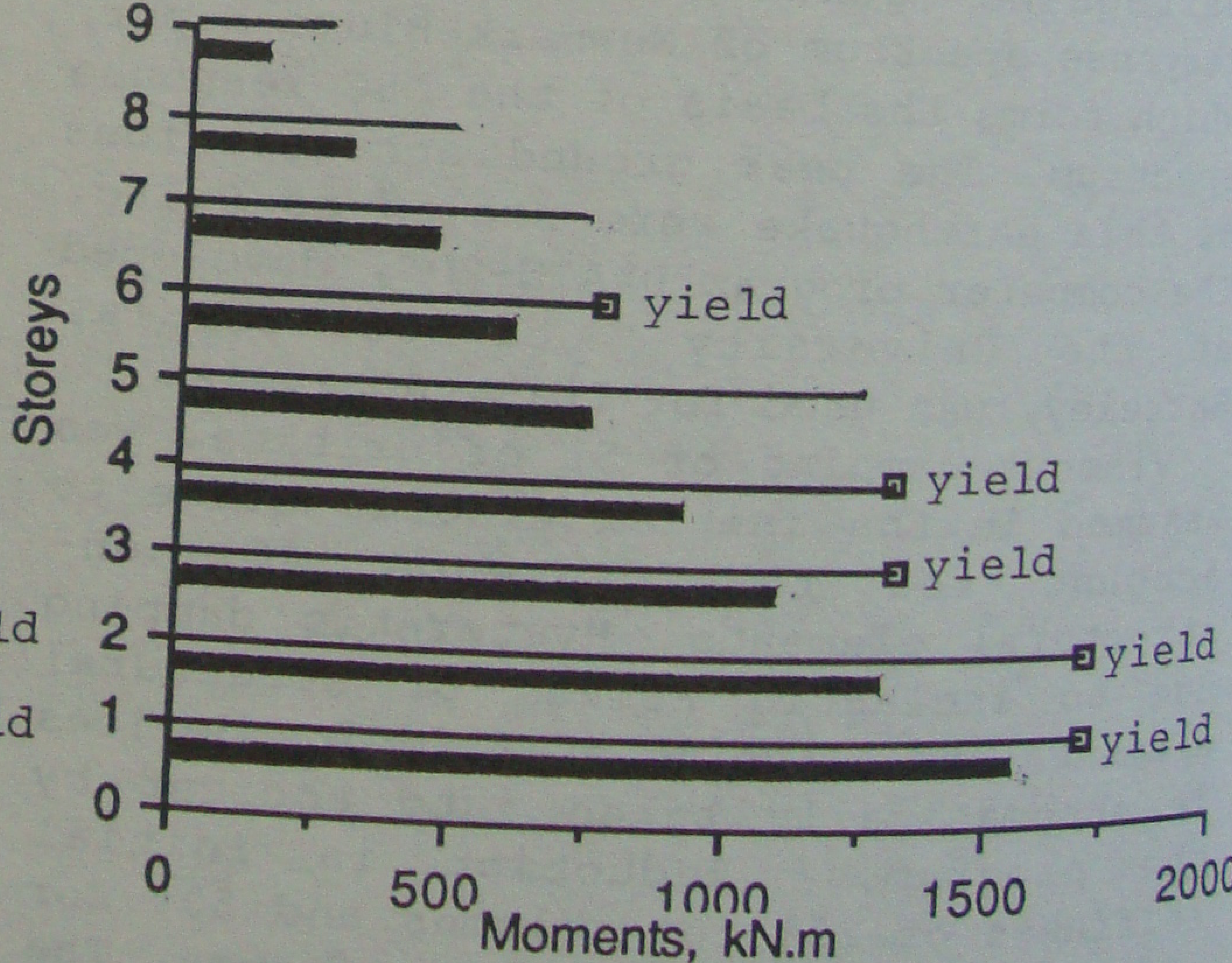


FIG. 16. SLAB MOMENTS (Nonlinear)

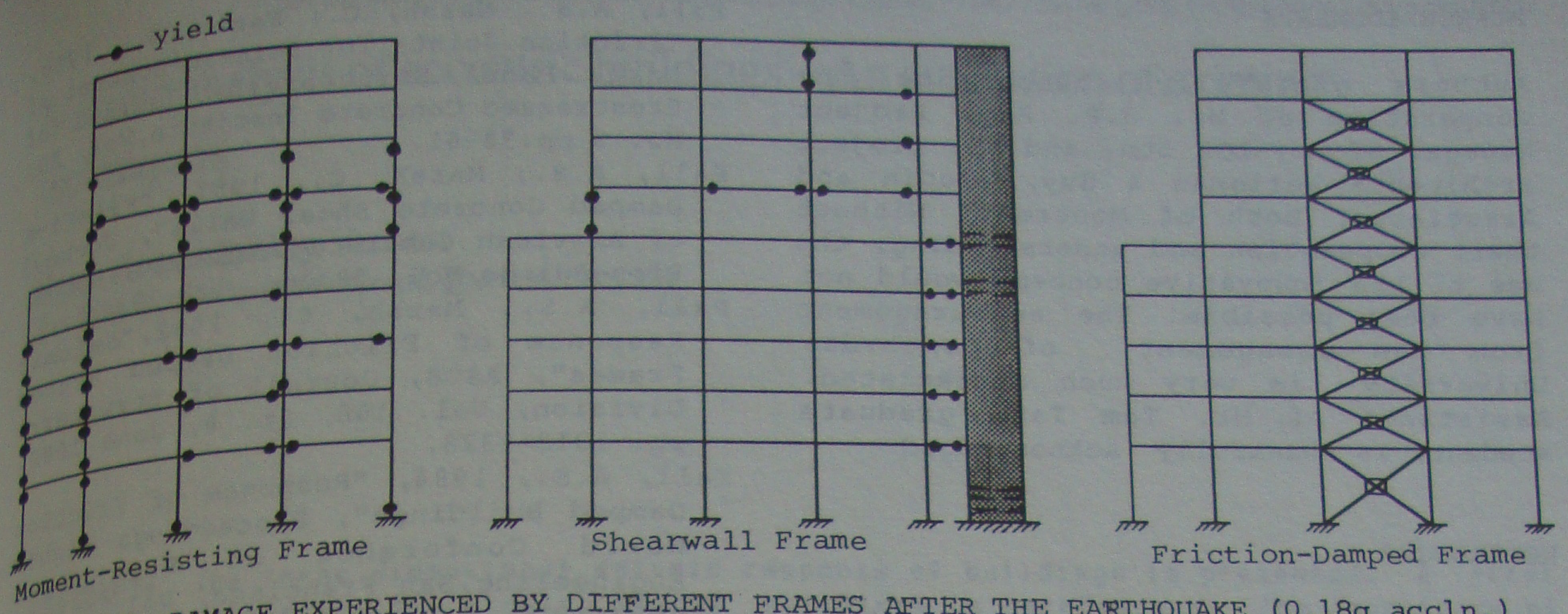


FIGURE 17. DAMAGE EXPERIENCED BY DIFFERENT FRAMES AFTER THE EARTHQUAKE (0.18g accln.)

5.Since the moment-resisting frame was badly damaged, another analysis was made using a further 25% reduction of the stiffness. The peak deflection at the top of the building was 355 mm (H/110) and the extent of the damage to the frame increased. Time histories of this analysis is shown in Figure 12.

6.The slip load of the friction device is about double the force developed in the bracing due to quasi-static earthquake loads. Hence, the devices will not slip during wind storms or moderate earthquakes but will go into action only during a major earthquake.

7.In order to quantify the performance of the FDF realtive to the MRF, an equivalent viscous damping study was made. Viscous damping was added to the MRF until the dynamic response of the MRF became equal to that of the FDF. This equality was achieved by introducing 52% damping. The percentage of damping increases as the earthquake intensity increases.

COST ANALYSIS

The main savings achieved through the use of device-equipped bracings was the elimination of concrete shear walls. Also, as the forces in the members are significantly reduced, it was possible to effect savings in the construction materials.

The elimination of shear walls around stair halls and elevators required the provision of additional columns, infilling with concrete masonry block walls on the interior side and covering the braced bays with gypsum board walls on the exterior side.

The other expenses were the cost of friction devices, the cost of steel bracing and cost of steel inserts to be embedded in the concrete columns for connecting steel bracings.

The estimate was based on the prevailing market rates. It was seen that the use of new concept offers a net saving of 6.5% of the structural cost or 1.5% of the total building cost. This is considered to be quite significant.It is expected that in regions more severely affected by the earthquakes the savings will be much higher. Also, when the codes are adequately revised to account for the energy dissipation capacity of the new concept, the level of savings will further increase.

CONCLUSIONS

The concept of friction-damped steel bracing in concrete frames is shown to provide a practical, economic and effective new approach to the problems of resisting seismic loads. It raises the level of earthquake resistance from the avoidance of collapse to the avoidance of damage. Some of the many advantages are:

1.Savings in the initial cost of construction.

2.Savings in the life cycle cost as the damage is minimized.

3.Added safety to the structure, the occupants and the contents.

4.Savings in insurance premiums, where applicable.

5.The devices can be conveniently incorporated in existing frame buildings to upgrade their seismic resistance.

ACKNOWLEDGMENTS

Authors gratefully acknowledge the cooperation of Mr. J.P. Roy, Project Manager of Roy-LGL Ltd. and the project architects Werleman & Guy, Blouin and Associates, both of Montreal. Without their cooperation and understanding, the use of the innovative concept would not have been possible. The encouragement from the management of Concordia University is very much appreciated. Assistance of Mr. Tom Tan, graduate student, is thankfully acknowledged.

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