SEISMIC EVALUATION AND RETROFIT OF INDUSTRIAL BUILDINGS

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SUMMARY

Performance based design in its actual format is certainly a comprehensive tool for seismic risk minimisation. What needs to be kept in mind however is that unless structural engineers work hand in hand with other discipline engineers, contractors and manufacturers, the implementation of performance based design will be hard. Also, the understanding of non-linear analytical methods, procedures and the inclusion of cumulative damage methods is of prime importance for good implementation of performance based design.

The use of non-conventional methods, such as coupling building structures to non-building structures, disconnecting structures that induce mass and/or stiffness eccentricity, the addition of supplemental damping devices through the use of fluid dampers and fluid damper-springs to reduce the overall response and the use of snubbers to eliminate pounding between the buildings results in efficient retrofit schemes.

INTRODUCTION

Considerable changes occurred in the seismic requirements of the National Building Codes since their earliest editions. For instance, earthquake loads specified in the earliest editions of the National Building Code of Canada [NBC, 1970] are considerably lower than the 1990 and 1995 editions. Sometimes the ratio of base shear of the latest editions can be as high as twice that of the earliest editions. This often means that buildings built back then had, in most instances, wind loads governing the design process. It should also be noted that industrial buildings receive, in general, numerous structural modifications over the years. In some instances, it has been observed that bracing members were removed to accommodate equipment. In other instances, the basic system is modified to accommodate specific production requirements; example of this type of interventions is the removal of a building column for a larger entrance. Other examples of important modifications show up in the form of additional substantial mass being installed in the structures such as new conveyors, new bunkers, and new heavy equipment etc. All the above leads to a modified structure with geometrical, stiffness and mass distribution characteristics that are totally different from the structure originally designed.

Existing buildings have been shown to sustain significant over-stress, local, and global failures when subjected to moderate and strong intensity earthquakes. The advent of recent earthquakes such as the 1989 Loma Prieta (California), the 1994 Northridge (California) and the most recent 1995 Hyogo-ken Nanbu (Japan) earthquakes showed that steel buildings are still vulnerable to moderate and strong earthquakes. The 1995 Hyogo-ken Nanbu earthquake produced a large number of damaged steel buildings. In fact, 1247 steel buildings were reported to be damaged, of which 286 had collapsed or were in danger of collapsing during aftershocks [Tremblay et al., 1996].

PERFORMANCE LEVELS AND OBJECTIVES RELATED CONSIDERATIONS

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Performance based design has been in use in various engineering disciplines and sectors such as manufacturing and to a limited extent in structural engineering for many years now. In fact numerous design codes have used and are still using one form or another of performance based design. Within the context of a limit state design, two levels of designs exist: a) the first level consists of serviceability limit state, and b) the second level covers the member strength limit state. The advent of recent strong earthquakes brought Performance Based Design to the forefront. Performance Based Design and its performance levels and corresponding objectives are well covered in documents such as ATC-40, FEMA-273 and Vision 2000 of the “Bleue Book”.

The following table shows performance levels and the corresponding basic objectives, essential/hazardous objective and safety critical objective levels associated with performance based design:

Event	Recurrence Interval	Probability of Exceedence	Fully Operational	Operational	Life Safe	Near Collapse
Frequent	43 years	50% in 30 years	●	❍	❍	❍
Occasional	72 years	50% in 50 years	■	●	❍	❍
Rare	475 years	10% in 50 years	❖	■	●	❍
Very rare	970 years	10% in 100 years		❖	■	●

Where:

● : is for basic objective

■ : is for Essential/Hazardous Objective

❖ : is for Safety Critical Objective

From an industrial building owner’s point of view, the major objective is first to protect people within and around the facility, and then protect the structure for various performance levels. This later objective in most cases would span from the Fully Operational level to the Near Collapse level. This should be achievable when planning for a new facility, and would only account for a small percentage of the capital cost. For an existing structure, this line of thinking will in most cases not be achievable. Protecting against the full spectrum of performance levels will either be too stringent, exhaustively expensive or not practical since it will hinder production or operation in the facility. The project team should thus put together a list of priorities that could include structural considerations as well as non-structural considerations. For instance, a chemical tank falling and spilling its content might be more dangerous than the consequences of a collapse of a member, which has no bearing on the overall structural system stability.

For life safety, the collapse performance level is evidently to be achieved at all cost. Also to save life, the structural engineer should closely work with other engineers to ensure that services inside an industrial building are designed and manufactured to behave adequately for a specified performance level. For instance, what could be a tolerable drift from a structural point of view, could also be an intolerable movement for a Carbon Monoxide pipe that could crack and lead to potentially disastrous consequences. A minor settlement that is acceptable from a structural point of view may not be acceptable to an in-ground water pipe that is cooling a furnace.

Low cycle fatigue due to cumulative damage and its consequent cracks in connections of moment resisting frames could have equal disastrous consequences from a strong motion earthquake compared to consequences from a repeated number of moderate motion earthquakes. For this reason, models for cumulative damage assessment [Daali & Korol, 1995, and Daali, 1995] should be part of non-linear static analysis and non-linear dynamic time-history analyses.

STATIC AND DYNAMIC ANALYSES

Often times, the evaluation and assessment of existing industrial building starts with a visual study of drawings or the structure itself followed with rule-of-thumb rough checks. The basic objective in this phase is to identify the system weaknesses such as lack of secondary load paths, lack of effective systems (bracing) that could transfer loads to the foundations, distribution of mass and stiffness.

In consulting offices, it is accepted practice to design new structures and evaluate/assess existing structures using two and three-dimensional equivalent static analyses. The analysis uses code specified lateral loads applied to a structure modelled with an elastic linear stiffness. The results of this linear elastic static analysis will be internal forces and displacements whose magnitudes are approximately those of the design earthquake. However, it should be mentioned that the nature of industrial type of structures presents plane and vertical irregularities that makes the use of the static analysis inappropriate in most cases.

For the reasons above mentioned linear time history dynamic analysis and linear response spectrum analysis found their way in design offices. To use these two methods, the structure is also modelled with an elastic linear stiffness. The advantage of these methods of analyses over the static linear analysis resides in a better distribution of forces that takes into account system irregularities and mass distribution. But the shortcomings reside in the fact that material and geometrical non-linear effects and cumulative damage are not included.

Since the advent of performance based design, non-linear static (or pushover) analysis and dynamic analysis became the tools of choice for analysis and evaluation for the identification of areas where there is a lack of stiffness and ductility, or weaknesses in the member or connection. The FEMA-273 and ATC-40 are two documents where modelling techniques, guidelines for use of ground histories and acceptance criteria can be found. The static non-linear procedure is used on structural models with non-linear moment-rotation (or force-deformation) relationships. Gravity loads are first applied, then the lateral loads are progressively increased and a load-deformation envelope for the whole structure is obtained. The non-linear time history dynamic analysis is similar to the linear time history dynamic analysis except that non-linear force-deformation relationship are included in the modelling and that cumulative damage models are also ideally included.

STATE-OF-THE-ART

The relationship governing the conservation of energy [Uang and Bertero, 1988] dictates that the energy input resulting from a strong motion earthquake is absorbed through the sum of the recoverable elastic energy, the kinetic energy, the damping energy and the irrecoverable hysteretic energy. If a structure does not have the ability to dissipate energy through hysteretic energy, then it should either have sufficient strength to resist the earthquake forces or have adequate damping to dissipate the input energy

Supplemental damping can be introduced in a structure by the use of mechanical devises. The latter dissipate energy through friction, yielding of specific components, motion of a piston into a fluid or a material which is similar or by viscoelastic action. To achieve this, numerous devices can be found in the market. In fact as early as the 1890’s fluid dampers have been used to attenuate recoil in artillery guns [see Taylor and Constantinou, 1994]. Other examples of dampers can be found in the friction-damper device proposed by Pall and March in 1982, the Taylor fluid dampers which have been in use since 1955, in commercial and aerospace/defense projects and the Jarret viscoelastic elastomers dampers.

RETROFIT SCHEMES

The preliminary assessment and evaluation of existing buildings dating from 30 years and older often reveals that the lateral resistance of the structural system is only a small portion of the code base shear loads. Furthermore, considerable irregularities would also exist in mass and stiffness distribution.

To enhance the lateral resistance of an existing industrial building, very often it will be a negotiating process whereabouts the structural engineer proposes retrofit schemes and receives feedback from the plant management and production people. The process is not easy and would require a lot of imagination.

For instance a continuous multi-bay crane runway could be rigidly linked, tied with dampers or snubbers to the main structure. Two parts of the building separated by an expansion joint could also be linked together with snubbers and dampers strategically placed to force the parts of the building to move in-phase and thus avoid out-of-phase motion.

Among other methods of retrofit for existing industrial buildings could be the addition of external structures that are in line or enveloping the existing ones. Also, a modification of shear connections into moment resisting connections in girder to column connections would help in securing alternative load paths. The addition of bracing coupled with friction or fluid dampers would definitely be of great help in dampening the earthquake motion.

For industrial buildings where a section of the building induces high eccentricity in mass and rigidity, it might be beneficial to disconnect or isolate the later from the former. Particular attention should be paid to the retrofit of structures built of heavy clay brick masonry walls with a soft and weak first story. At first glance, the use of stiff bracing seems to be a suitable solution to brace the first story bays. This, however, would create a strong first story susceptible to transfer higher forces to the higher story, eventually damaging the clay brick masonry walls. Although rods and flat bars tension-only bracing are known to exhibit a rather poor behaviour in older buildings, tension-only bracing if properly detailed can behave in a satisfactory manner. One such example of such behaviour is the tension-only bracing used in a building, which was subjected to the recent Hyogo-Ken Namibia earthquake [Tremble et al. 1996].

CASE STUDY

When the original smelter building in province of Quebec, Canada was built in 1952, it accommodated five furnaces and was 420 feet (128 m) long. As the requirements for steel production grew, the building was gradually extended so today it houses nine electric arc furnaces and is about 1000 feet (305 m) long. The building is 110 feet (34 m) wide and 70 feet (21 m) high and is separated in longitudinal direction by a 2-inch (5-cm) expansion joint. Crane aisle on West Side and furnace aisle on the East Side run for the whole length of the building. Furnace aisle accommodates the furnaces, electrode floor, and materials handling, such as conveyor galleries and bunkers. Lateral loads are resisted by moment resisting frames in the transverse direction and by a braced system in the longitudinal direction.

The crane runway consists, longitudinally, of two continuous moment resisting frames, separated by a two-inch (50-cm) expansion joint. The crane runway is attached to the main structure transversally above the height of the crane columns by a series of shear links.

Attached to the building along the long side are transformer houses constructed of a steel skeleton frame infilled with concrete masonry blocks in the first story and a one-foot thick clay brick masonry walls in the stories above.

Static and three dimensional linear dynamic analyses were first used to evaluate the resistance of the smelter building when subjected to seismic lateral loads determined per the Canadian Building Code [NBC 1995]. This was followed with non-linear push over and times history analyses. The evaluation of the building concluded that the building could resist the NBC seismic loads in the transverse direction but was significantly weak in the longitudinal direction. Only 36% of the seismic loads could be resisted in the longitudinal direction. Several weaknesses that could possibly lead to a partial or total collapse of the building were identified. The most important among these were the lack of secondary load paths, the lack of ductility, lack of effective systems (bracing) that could transfer loads to the foundations and finally various local weaknesses such as absence of moment connections.

An independent reviewer specialising in the field of earthquake engineering first reviewed the studies performed by the consulting team. Then, within the context of performance based design approach, design levels and performance objectives were targeted and numerous retrofit schemes were reviewed for implementation. Finally, the upgrading option that was selected included the following items:

- Addition of a new tower coupled with the original building,

- Disconnection of the transformer houses from the smelter building,

- Modification of shear connections into moment connections,

- Connecting the independent crane runway framing system to building column lines, and

- Addition of vertical bracing.

To further enhance the response of the smelter building under earthquake forces, it was decided to introduce supplemental damping devices coupled with bracing system in the critical areas of the building. To eliminate "pounding" of the north and south parts of the smelter building, snubbers and damper units were placed into the building expansion joints. The latter were subjected to a rigorous testing program (Fig. 1) in the manufacturing plant before being shipped and installed in the building.

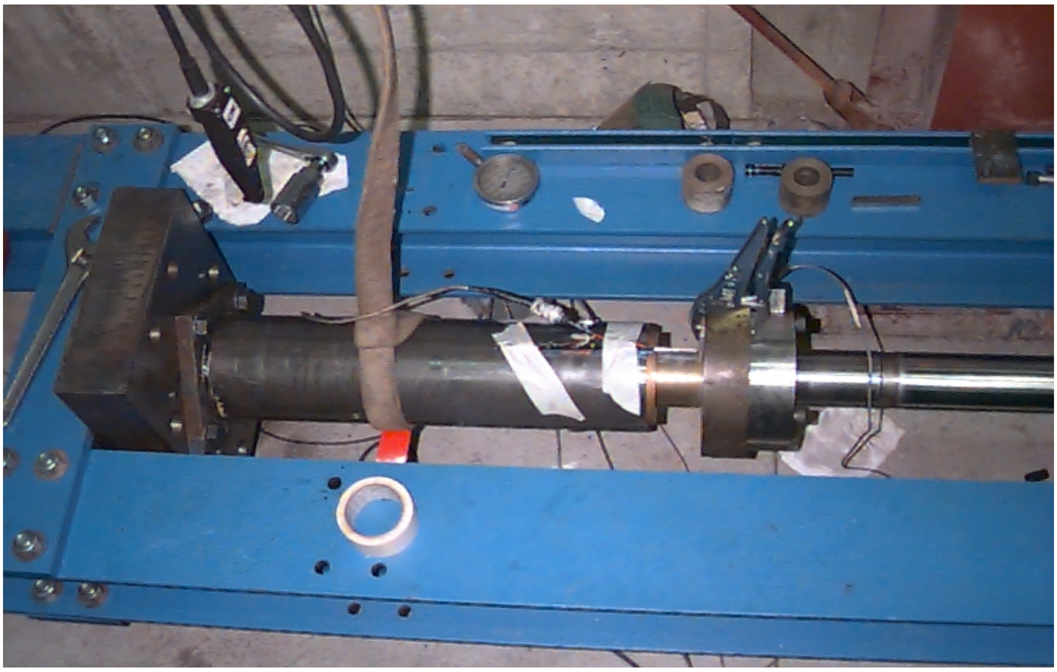

Fig. 1 Testing of a damper for a smelter building.

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