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Optimum design of base isolated RC structures



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Optimum design of base isolated RC structures



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ABSTRACT

The main idea of the study is the optimum design and the economic evaluation of reinforced (RC) conventional and isolated structures. For the purpose of the study two symmetrical RC structures were studied, designed both with and without seismic isolation, following a performance based concept. The seismic isolation was accomplished by the use of Lead-Rubber Bearings (LRB) and High Damping Rubber Bearings (HDNR).

In the first chapter, the seismic isolation technique is described, as well as the conditions and the applications of the method worldwide, along with the types of the isolation devices.

In the second chapter, the modeling, the preliminary design and the final design of the bearings is described.

In the third chapter, the analysis procedures are presented, and specifically the Linear Static Procedure (LSP), the Nonlinear Static Procedure (NSP) according to the recommendations of FEMA-356 and the Nonlinear Dynamic Procedure (NDP).

Thereafter, in the fourth chapter the structural optimization problem is described, along with the history of the technique and the formulation of the problem. The design variables, the objective function and the constraint functions are defined, as well as the three types of optimization. Finally, the Evolutionary Algorithms (EA) are presented, with emphasis to the Differential Evolution (DE).

In the fifth chapter the procedure of Life-Cycle Cost Analysis (LCCA) is presented, which can be used as an assessment tool of the response of the building during its expected lifetime. The calculation of the procedure is analyzed, as well as the steps to incorporate the nonlinear dynamic analysis in the calculation procedure.

Subsequently, in the sixth and seventh chapter the test cases of the study are presented analytically, along with the conclusions that are obtained from this process.

ΠΕΡΙΛΗΨΗ

Ο κύριος στόχος της παρούσας μελέτης είναι ο βέλτιστος σχεδιασμός και η αποτίμηση του κόστους συμβατικών και σεισμικά μονωμένων πολυώροφων κτιρίων από οπλισμένο σκυρόδεμα. Για το σκοπό της εργασίας μελετήθηκαν δύο συμμετρικά κτίρια, ένα τριώροφο και ένα εξαώροφο, σχεδιασμένα και τα δύο με και χωρίς σεισμική μόνωση, ακολουθώντας τη διαδικασία αντισεισμικού σχεδιασμού με βάση την επιτελεστικότητα. Η σεισμική μόνωση υλοποιήθηκε με τη χρήση Ελαστομεταλλικών Εφεδράνων με Πυρήνα Μολύβδου (LRB) και με Ελαστομερή Εφέδρανα Υψηλής Απόσβεσης (HDNR).

Στο πρώτο κεφάλαιο περιγράφεται η τεχνική της σεισμικής μόνωσης, όπως επίσης και οι προτεινόμενες συνθήκες και εφαρμογές της μεθόδου παγκοσμίως, καθώς και οι τύποι των σεισμικών μονωτήρων.

Στο δεύτερο κεφάλαιο περιγράφεται η μοντελοποίηση, η προδιαστασιολόγηση, καθώς και ο τελικός σχεδιασμός σεισμικά μονωμένων κτιρίων.

Στο τρίτο κεφάλαιο παρουσιάζονται οι μέθοδοι ανάλυσης και συγκεκριμένα η Γραμμική Στατική Ανάλυση (LSP), η Μη Γραμμική Στατική Ανάλυση (NSP) σύμφωνα με τις συστάσεις του FEMA-356 και η Μη Γραμμική Δυναμική Ανάλυση (NDP).

Στη συνέχεια στο τέταρτο κεφάλαιο περιγράφεται ο βέλτιστος σχεδιασμός κατασκευών, όπως επίσης οι μέθοδοι επίλυσης τέτοιων προβλημάτων και η διατύπωση τους. Ορίζονται οι μεταβλητές σχεδιασμού, η αντικειμενική συνάρτηση και οι περιορισμοί του προβλήματος και τέλος, παρουσιάζονται οι Evolutionary Algorithms (EA), με έμφαση στη μέθοδο Differential Evolution (DE).

Στο πέμπτο κεφάλαιο παρουσιάζεται η μέθοδος αποτίμησης του κόστους κύκλου ζωής μιας κατασκευής (LCCA), η οποία βασίζεται στην απόκρισης μιας κατασκευής στη διάρκεια ζωής της. Περιγράφονται τα βήματα της μεθόδου και ο τρόπος εφαρμογής της μη γραμμικής δυναμικής ανάλυσης στον υπολογισμό του κόστους κύκλου ζωής.

Τέλος, στο έκτο και έβδομο κεφάλαιο παρουσιάζονται αναλυτικά οι εφαρμογές της παρούσας εργασίας, όπως επίσης και τα συμπεράσματα που προέκυψαν από τις αναλύσεις.

To my friends and family

"Always desire to learn something useful" Sophocles

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CHAPTER 1

SEISMIC ISOLATION SYSTEMS

CHAPTER 1

1.1 INTRODUCTION

An earthquake causes all buildings to be shaken by the ground. Buildings that are shorter and/or stiffer amplify the ground motions and experience accelerations that are much larger than actual ground acceleration. The geometry and stiffness characteristics of the building also cause amplification of the ground motion up through the building.

Seismic isolation may be an effective rehabilitation strategy if the results of seismic evaluation show deficiencies attributable to excessive seismic forces or deformation demands, or if it is desired to protect important contents and nonstructural components from damage. A seismically isolated structure uses seismic isolation devices which increase the period of shaking of a building. They are inserted between the building and ground in order to reduce the amplification of the earthquake motion in the building, thus mitigating the shaking of the building.

Qualitatively, a conventional structure experiences deformations within each story of the structure (i.e., interstory drifts) and amplified accelerations at upper floor levels. On the contrary, isolated structures experience deformation primarily at the base of the structure (i.e., within the isolation system) and the accelerations are relatively uniform over the height.



Figure 1.1 Behavior of building structure with and without base isolation system

Seismic isolation devices are most effective when used in structures on stiff solid and structures with low fundamental period (low-rise building). Stiff structures are particularly well-suited to base isolation, since they move from the high acceleration region of the design spectrum to the low acceleration region. In addition, for very stiff structures, the excitation of higher mode response is inhibited, since the superstructure higher mode periods may be much smaller than the fundamental period associated with the isolation system.

Softer soils tend to produce ground motion at higher periods which in turn amplifies the response of structures having high periods. Thus, seismic isolation systems, which have a high fundamental period, are not well-suited to soft soil conditions.



Figure 1.2 Effect of soil conditions on isolated structure response

The motivation factors for applying seismic isolation to retrofit projects are, at first, to minimize the modification/destruction of the building (historical building preservation), to maintain the functionality of the building after an earthquake, to provide a more economic design solution than the usual method, since the long-term economic loss is reduced, and finally to protect the content, since the value of content may be greater than the structure (i.e., museums, galleries, etc.).

1.2 SEISMIC ISOLATION BUILDINGS

The first evidence of architects using the principle of base isolation for earthquake protection was discovered in Pasargadae, a city in ancient Persia, now Iran, back to 6th century BC.

Although the first patents for base isolation were in the 1800's, and examples of base isolation were claimed during the early 1900's (e.g. Tokyo Imperial Hotel) it was the 1970's before base isolation moved into the mainstream of structural engineering. Isolation was used on bridges from the early 1970's and buildings from the late 1970's. Bridges are a more natural candidate for isolation than buildings because they are often built with bearings separating the superstructure from the substructure.

1.2.1 Base-isolated buildings in the United States

The first base-isolated structure to be built in the United States was the Foothill Communities Law and Justice Center (FCLJC), located in the city of Rancho Cucamonga, east of downtown Los Angeles. Not only was it the first base isolated building in the United States, it was also the first building of the world to use isolation bearings made of high-damping natural rubber.



Figure 1.3 Foothill Communities Law and Justice Center, Rancho Cucamonga, California

The same high-damping rubber system was adopted for a building commissioned by Los Angeles Country, the Fire Command and Control Facility (FCCF). The FCCF houses the computer and communications systems for the fire emergency services program of the country and is required to remain functional during and after an extreme earthquake. This building was isolated based on a comparison of conventional and isolated schemes designed to provide the same degree of protection. On this basis the isolated design was estimated to cost 6% less than the conventional design.

Other base-isolated buildings in the United States are the Emergency Operations Center (EOC) in Los Angeles and the Traffic Management Center for Caltrans in Kearny Mesa, California, near San Diego. Other base-isolated building projects in California include a number of hospitals, such as M.L. King/C.R. Drew Diagnostics Trauma Center in Willowbrook.

In addition to the new buildings described above, there are a number of very large buildings in California that were retrofitted using base isolation. The retrofit of the Oakland City Hall was completed in 1995 and the retrofit of the San Francisco City Hall in 1998.





Figure 1.4 Oakland City Hall

Figure 1.5 San Francisco City Hall

1.2.2 Base-isolated buildings in Japan

Earthquake-resistant design has always been a high priority in Japan, and many mechanisms for the seismic-protection of structures, including forms of seismic isolation, have been developed there. Japanese structural engineers generally design buildings with more seismic resistance than do U.S. or European engineers and are willing to consider more costly designs.

All base isolation projects in Japan are approved by a standing committee of the Ministry of Construction. As many of the completed buildings have experienced earthquakes, in some cases it has been possible to compare their response with adjacent conventionally designed structures. In every case where such a comparison has been made, the response of the isolated building has been highly favorable, particularly for ground motion with high levels of acceleration.

One of the largest base-isolated buildings in the world is the West Japan Postal Computer Center located in Sanda, Kobe Prefecture. The use of isolation in Japan continues to increase, especially in the aftermath of the Kobe earthquake. As a result of superior performance of the West Japan Postal Computer Center, there has been a rapid increase in the number of permits for base-isolated buildings, including many apartments and condominiums.

1.2.3 Base-isolated buildings in New Zealand

The first base-isolated building in New Zealand was the William Clayton building in Wellington. Completed in 1981, it was the first building in the world to be isolated on lead-rubber bearings. Other seismic-isolated buildings are the Union House, Auckland,

the Wellington Central Police Station and the National Museum of New Zealand, Wellington.

1.2.4 Base-isolated buildings in Europe

In Europe, base isolation has been studied most actively in Italy under the auspices of the National Working Group on Seismic Isolation [Gruppo de Lavoro Isolamento Sismico (GLIS)]. One of the buildings that have been constructed using base isolation in Italy is the Administration Center of the National Telephone Company (SIP), a complex of five seven-story buildings in Ancona.

In Greece until now seismic isolation is performed in bridges (i.e. Isthmus of Corinth) and in structures with vital significance which are required to remain functional during and after an extreme earthquake, such as Greece's centralized liquefied natural gas (LNG) storage tanks in Revinthousa Island, near Athens. These tanks contain 38 million gallons of flammable LNG and are situated within one of Europe's highest seismic regions. The bearing performance requirements for this project were the most stringent in the history of seismic isolation. The bearings were required to maintain their design properties while fully accommodating the effects of: 35 years of aging in a marine environment; simultaneous lateral and vertical earthquake motions; temperatures ranging from -12°C to 30°C. Friction bearings were selected over elastomeric bearings after tests of full size bearings that showed that they were best able to satisfy these demanding performance requirements and would thereby achieve the safest tank performance.

Another building that designed with seismic isolation technique in Greece was the Onassis House of Letters and Arts, which is a R/c structure having unique shape and dynamic behavior. In order for structural design to meet the high performance seismic specifications that were set, seismic isolation should be used. Isolators of friction-pendulum (FPS) type were selected and placed under the ground floor slab, due to reduced cost and construction effectiveness.



Figure 1.6 Onassis House of Letters and Arts, Athens, Greece

Finally, seismic isolation was judged to be the most appropriate method applied for the protection of the new Acropolis Museum, located at the southern edge of Acropolis. The significance of the building, along with the enormous historical value of the exhibits leaves no potentials for any kind of damage at the frame of the building, while the architectural design foresaw large open spaces in the interior, so as to provide uninterrupted view of the Parthenon. The seismic isolation system consists of 94 pendulum devices. The level of seismic isolation is installed underneath a concrete base plate dimensioned 110m×70m, on which the 40m high four storey building is constructed. The bearings are designed to undertake vertical load 16000kN and have the maximum horizontal displacement +/-250mm.



Figure 1.7 Acropolis Museum, Athens, Greece

1.3 TYPES OF ISOLATION DEVICES

The two basic types of isolation bearings are *sliding* and *elastomeric (rubber)* bearings. Typically, isolation systems consist of either elastomeric bearings alone or sliding bearings alone, although in some cases they have been combined.



Figure 1.8 Sliding bearing



Figure 1.9 Elastomeric bearing

1.3.1 Sliding bearings

Sliding systems are simple in concept and have a theoretical appeal. A layer with a defined coefficient of friction will limit the accelerations to this value and the forces

which can be transmitted will also be limited to the coefficient of friction times the weight.

Sliders provide the three requirements of a practical system if the coefficient of friction is high enough to resist movement under service loads. Sliding movement provides the flexibility and the force-displacement trace provides a rectangular shape that is the optimum for equivalent viscous damping. Sliding bearings typically utilize either spherical or flat sliding surfaces.

 Pure Friction Systems: It is the earliest and simplest sliding isolation scheme and best represents the principles of sliding isolation systems. The system utilizes a sliding joint to decouple the superstructure from the substructure and operates under the principle of sliding friction. At low lateral service loads, the entire structure acts as a fixed-base system, since lateral forces are too insignificant to overcome the static frictional force and induce horizontal displacement. When the system is subjected to significant lateral seismic forces, the frictional force is overwhelmed and sliding is mobilized. Accelerations in the structure are reduced through the dissipation of energy through friction in the form of Coulomb damping. The lateral force required to overcome the static frictional force is a function of the coefficient of static friction and can be controlled through the selection of material to be employed at the bearing surface.

Clear disadvantages of the system include continuous maintenance of the bearings to ensure a constant coefficient of friction and the inability of the system to recenter after an extreme event.

Friction Pendulum System bearing (FPS): It is the most widespread sliding seismic isolation bearing in use within the United States. FPS uses geometry and gravity to achieve the desired seismic isolation results. The FPS concept is based on an innovative way of achieving a pendulum motion. It combines the concept of sliding isolation systems with the action of a pendulum. The superstructure is isolated from the substructure via a bearing that is comprised of an articulated slider resting on top of a convex bearing surface with a low coefficient of friction, usually made of chrome or stainless steel. When lateral seismic forces overcome static friction the articulated slider is displaced along the convex spherical bearing surface. If friction between the articulator and the bearing surface is neglected, the system behaves as a simple pendulum. The restoring force that recenters the friction pendulum systems provided by the change in direction of the frictional and normal forces as the articulator slides up the wall of the curved bearing surface. Coulomb damping generated through sliding friction provides constant energy dissipation in the bearing. The effective stiffness and

subsequent shifted period of the isolation system, based on dynamics of a pendulum, is dependent upon the radius of curvature of the convex bearing surface. This kind of bearings was used in Onassis House of Letters and Arts and in Acropolis Museum of Athens.



Figure 1.10 Friction Pendulum System Bearing (FPS)

Sliding Isolation Pendulum Bearings (SIP): SIP bearings can be compared to a spherical bearing that can move in all directions. The horizontal displacements caused by seismic events are accommodated by the sliding movement and in the same time the energy that is introduced is converted either into heat or into potential energy. It also provides recentering to the superstructure by means of its dead weight into the central position of the curved sliding surface. Therefore, SIP-bearings combine the four main requirements of the seismic isolation: Vertical load transmission, horizontal displacement, energy dissipation and recentering.



Figure 1.11 Sliding Isolation Pendulum Bearings (SIP)

• *Resilient Friction Base System (R-FBI)*: It attempts to overcome the problem of the high friction coefficient of Teflon on stainless steel at high velocity between the top and bottom of the bearing is divided by the number of layers, so that the velocity at each face is small, maintaining a low friction coefficient. In addition to the sliding elements, there is a central core of rubber that carries no vertical load but provides a restoring force. A central steel rod was inserted in the rubber core to improve the distribution of displacement among the sliding layers.



Figure 1.12 Resilient Friction Base System (R-FBI)

1.3.2 Elastomeric (rubber) bearings

Elastomeric bearings consist of a series of alternating rubber and steel layers. The rubber provides lateral flexibility while the steel provides vertical stiffness. In addition, rubber cover is provided on the top, bottom, and sides of the bearing to protect the steel plates. In some cases, a lead cylinder is installed in the center of the bearing to provide high initial stiffness and a mechanism for energy dissipation.

Natural rubber bearings were first used for the earthquake protection of buildings in 1969 for the Pestalozzi School in Skopje. Characteristic of isolation systems of this kind, the horizontal motion is strongly coupled to a rocking motion, so that purely horizontal ground motion induces vertical accelerations in the rocking mode.

• Low-Damping Natural or Synthetic Rubber Bearings: The isolators have two thick steel endplates and many thin steel shims. The rubber is vulcanized and bonded to the steel. The steel shims prevent bulging of the rubber and provide a high vertical stiffness but have no effect on the horizontal stiffness, which is controlled by the low shear modulus of the elastomer. The material behavior in

shear is quite linear up to shear strains above 100%, with the damping in the range of 2-3% of critical.

The advantages of the low-damping elastomeric laminated bearings are many: They are simple to manufacture, easy to model and their response is not strongly sensitive to rate of loading, history of loading, temperature and aging.

The primary disadvantage of natural rubber bearings is the necessity for auxiliary damping devices. They are considered low-damping devices because they exhibit relatively small damping values of approximately 2-3% of critical damping. Damping can be controlled to a limited extend by enhancing the material properties of the elastomer, but usually supplementary external damping devices, such as viscous dampers and hysteretic dampers, must be used in parallel with the bearings to aid in the control of motion under both low level service loads and extreme seismic loads.



Figure 1.13 Low-Damping Synthetic Rubber Bearing

 High-Damping Natural Rubber Bearings (HDNR): In order to eliminate the need of supplementary damping elements, it was developed a natural rubber compound with enough inherent damping. The damping's shear modulus is 0.35-1.4 MPa, its maximum shear strain is 200 to 350%, while the damping values range between 7-14% of the critical.

The dynamic properties of high damping rubber bearings tend to be strongly sensitive to loading conditions. For example, high damping rubber bearings are subjected to scragging. Scragging is a change in behavior (reduction in stiffness and damping) during the initial cycles of motion with the behavior stabilizing as the number of cycles increases. The behavior under unscragged (virgin) conditions can be appreciably different from that under scragged (subjected to strain history) conditions. Over time (hours or days), the initial bearing properties are recoverable. Lead-Rubber Bearings (LRB): This kind of seismic isolator was invented in 1975 in New Zealand by Bill Robinson and is used extensively in New Zealand, Japan and the United States. Their structure is similar to low-damping rubber bearings, but they contain a central lead plug which increases the initial stiffness of the bearing, as it provides wind loading restraint, and increases the energy dissipation capacity of the bearing. After the lead yields, it dissipates energy as it is cycled. Fatigue of the lead is not a concern, since lead recrystallizes at normal temperatures.

The damping's shear modulus is 0.525-0.7 MPa, its maximum shear strain is 125 to 200% and the lead yield stress is about 1500 psi. Normally, the diameter of the central lead plug is 15-33% of the overall diameter of the bearing.

Because it incorporates a damping, it has all four of the functions necessary for a seismic isolation device:

- 1. An isolation function to carry the weight of the building while allowing it to move freely in the horizontal direction,
- 2. A restoring mechanism to return the building which moved in the horizontal direction to its original position,
- 3. A damping function which absorbs the energy of the earthquake and attenuates the building's shaking,
- 4. An initial strength function which prevents the building from moving when it is subjected to forces such as wind.



Figure 1.14 Hysteresis of a Lead-Rubber Bearing (LRB)

CHAPTER 1









1.4 RETROFIT OF EXISTING BUILDINGS

Retrofit of existing buildings to improve their earthquake safety involves additional considerations, compared with new construction, because of the constraints already present. Some structures are inherently more suitable for retrofit using seismic isolation than others. Buildings are often difficult to retrofit. However, seismic isolation may often be an effective solution for increasing the earthquake safety of existing buildings without the addition of new structural elements which detract from the features which originally make the building worth preserving. Although seismic isolation reduces earthquake forces, it does not eliminate them. Consequently, the strength and ductility of an existing structure must at least be sufficient to resist the reduced forces that result from isolation. If the strength of the existing structure is extremely low (less than 0.05 of the weight of the building), then additional strengthening versus some strengthening and the provision of isolation will need to be studied.

The pros and cons with regard to the plane of isolation are included as follows:
- Any structure with a full subbasement or basement that can be temporarily disrupted is a good isolation candidate, since the work can be confined to that area.
- A structure with piled foundations can be more easily retrofitted at the foundation level than one with spread footings.
- Provisions for the zone of isolation at the top, bottom or mid-height of the basement, requires a detailed evaluation of the column capacities. If the strength of the column is not sufficient to resist the reduced isolation forces, three potential options exist: First, the column may be strengthened and act as a cantilever. Second, a new framing system with stiff beams may be developed at the plane of isolation to reduce the column forces. Third, the mid-height column solution may be considered, since it reduces the column moments significantly.



(3) Cut off Existing Column

(6) Cover for Fire-resistence

Figure 1.17 Reform work of mid-storey isolation

Normally, excavations are made around the building and the superstructure is separated from the foundations. Steel or reinforced concrete beams replace the connections to the foundations, while under those, the isolators replace the material removed. Careful attention to detail is required where the building interfaces with the ground, especially at entrances, stairways and ramps, to ensure sufficient relative motion of those structural elements. In Figure 1.17 the described process is illustrated for a reform work of mid-story isolation.

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CHAPTER 2

DESIGN OF ISOLATED STRUCTURES

CHAPTER 2

2.1 SEISMIC HAZARD LEVEL

The seismic criteria adopted by current model codes involve a two-level approach to seismic hazard:

- Design Basis Earthquake (DBE): That level of ground shaking that has a 10% probability of being exceeded in 50 years (475-year return period earthquake). It is described as a rare event.
- *Maximum Capable Earthquake (MCE)*: The maximum level of ground shaking that may ever be expected at the building site. This may be taken as that level of ground motion that has a 10% probability of being exceeded in 100 years (1000-year return period earthquake). It is described as a very rare event.

2.2 MODELING OF ISOLATION BEARINGS

In practice, all isolation bearings are modeled by a bilinear model based on the three parameters K_1 , K_2 and Q, as shown in Figure 2.1. The elastic stiffness for a monotonous loading K_1 is either estimated from available hysteresis loops from elastomeric bearing tests or as a multiple of K_2 , which is the post elastic stiffness for lead-plug bearings and friction pendulum bearings. The characteristic strength Q is estimated from the hysteresis loops for the elastomeric bearings. For lead-plug bearings Q is given by the yield stress in the lead and the area of the lead, while in the friction pendulum bearings it is given by the friction coefficient of the sliding surface and the load carried by the bearing. The postyield stiffness can be accurately estimated or predicted for all three types of bearings.

The non-dimensional characteristic strength is given by the following relation:

$$\alpha = \frac{K_1 - K_2}{K_2}$$
(2.1)

and is calculated through an iterative process.

The effective damping factor of the seismic base isolation system β_{eff} is defined by:

$$\beta_{eff} = \frac{4 \cdot Q \cdot (D_{target} - D_y)}{2 \cdot \pi \cdot K_{eff} \cdot D_{target}^2}$$
(2.2)

and takes values between 10% and 30%.

The effective period T_{eff} is calculated by the following relation:



Figure 2.1 Parameters of basic hysteresis loop

The effective stiffness of the LRB, defined as the secant slope of the peak-to-peak values in a hysteresis loop, is given by:

$$K_{eff} = \frac{4 \cdot \pi^2 \cdot m}{T_{eff}^2} = K_2 + \frac{Q}{D_{target}}, \quad D_{target} \ge D_y$$
(2.4)

where D_{y} is the yield displacement. In terms of the primary parameters,

$$D_{y} = \frac{Q}{K_{1} - K_{2}}$$
(2.5)

and the design displacement of LRB *D*_{target} is expressed as followed:

$$D_{target} = \frac{R_d}{4 \cdot \pi^2} \cdot T_{eff}^2 \begin{cases} D_D = \frac{R_{d(10\%/50)}}{4 \cdot \pi^2} \cdot T_D^2 & \text{for DBE} \\ D_M = \frac{R_{d(2\%/50)}}{4 \cdot \pi^2} \cdot T_M^2 & \text{for MCE} \end{cases}$$
(2.6)

To illustrate the effect of the selection of K_1 on the damping, consider a system with the same Q and K_2 values (thus the same effective period at all values of D and the same hysteresis loop) but modeled by different values of K_1 . Then:

 $K_1^1 = 51K_2$ Corresponding to a friction pendulum system

 $K_1^2 = 21K_2$ Corresponding to a lead-plug bearing

 $K_1^3 = 6K_2$ Corresponding to a high-damping rubber bearing

 $K_1^4 = 3K_2$ Another example of high-damping rubber bearing.

2.3 STEP-BY-STEP PROCEDURE FOR THE DESIGN OF ISOLATED STRUCTURES

2.3.1 Preliminary Design Steps

<u>Step 1</u>: Establish seismic zone factor Z.

Step 2: Establish site soil profile category.

<u>Step 3</u>: Calculate Maximum Capable Earthquake (MCE).

<u>Step 4</u>: Determine seismic coefficients according to the seismic zone factor and the site soil profile.

<u>Step 5</u>: Determine seismic coefficients according to the soil profile type determined in step 2.

<u>Step 6</u>: Determine structural system reduction factor R_i corresponding to the structural system used above the isolation interface from Table 2.1.

Construction	R 1
Special moment-resisting frame	2.0
Shear wall	2.0
Ordinary braced frame	1.6
Eccentric braced frame	2.0

Table 2.1 Reduction Factors for Isolated Construction

<u>Step 7</u>: Select the type of isolation bearings and the damping coefficients β_D and β_M (for LRB 15% - 35% and for HDNR 10%-20%).

<u>Step 8</u>: Select a desired isolated period of vibration T_D . Decide on an initial estimate for the isolated system fundamental period of vibration at the design basis displacement level, between 2.0 and 3.0 sec.

<u>Step 9</u>: Estimate the effective stiffness of the isolation system for the isolated period established in step 9.

<u>Step 10</u>: Estimate the minimum design displacement D_D, by the equation

$$D_D = \frac{(g/4\pi^2)C_{VD} \cdot T_D}{B_D}$$
(2.7)

and calculate the initial estimate of the minimum design displacement D_D .

Check: If this value is larger than what is acceptable for the project, go back to step 8 and start with a smaller estimate of the vibration period.

<u>Step 11</u>: Establish the minimum design lateral forces V_b and V_s , by the equations $V_b = K_{D,max} \cdot D_D$ (2.8)

$$V_s = \frac{K_{D,max} \cdot D_D}{R_I}$$
(2.9)

to estimate the minimum design lateral forces for the isolation systems and structural system at or below the isolation interface (V_b) and structural elements above the isolation interface (V_s), respectively.

Check: If the values of either V_b or V_s are larger than what is acceptable for the project, go back to step 8 and start with a larger estimate of the vibration period.

<u>Step 12</u>: Perform a preliminary design of the structural elements of the superstructure. With V_s estimated in step 11, static lateral forces at each level of the building are calculated. These lateral forces are used for preliminary stress sizing of superstructure elements based on drift limits ($0.010/R_1$ - static force procedure, $0.015/R_1$ - response spectrum analysis, $0.020/R_1$ - time history analysis).

Check: If the period of the fixed-base superstructure as designed is significantly different from that assumed in calculating the limitations on V_s in step 11, go to step 11 and verify the adequacy of V_s as assumed.

<u>Step 13</u>: Perform a preliminary design of isolator units and their distribution. Using the preliminary displacement, stiffness, force and damping properties established in the previous steps, design the isolator units to resist the gravity load, lateral load and displacement requirements.

2.3.2 Final Design Steps

<u>Step 14</u>: Construct mathematical model of the isolated structure. Incorporate the forcedisplacement characteristics of the isolation bearings obtained from step 13 in the models.

<u>Step 15</u>: Select an appropriate lateral response procedure.

<u>Step 16</u>: Finalize the target values of design displacements and isolated periods. Iteratively finalize the values of design displacement D_D' and maximum displacement D_M' for the project. $D_M' > D_D' > D_D$, where D_D was calculated in step 10. Establish the isolated period at design displacement and maximum displacement levels, T_D and T_M .

<u>Step 17</u>: Finalize the target values of effective stiffness, as follows:

$$K_{D,max} = K_{D,min} = \frac{DBE \ base \ shear}{D_D}$$
(2.10)

$$K_{M,max} = K_{M,min} = \frac{MCE \text{ base shear}}{D_M}$$
(2.11)

<u>Step 18</u>: Verify the effective period suggested by the mathematical model. Verify the effective periods T_D and T_M as determined by the mathematical model against those calculated by minimum values.

Step 19: Verify the damping level suggested by the Eq. (2.12), (2.13)

$$\beta_D = \frac{1}{2\pi} \left(\frac{\text{total area of hysteresis loop}}{K_{D,max} \cdot D_D^2} \right)$$
(2.12)

$$\beta_{M} = \frac{1}{2\pi} \left(\frac{\text{total area of hysteresis loop}}{K_{M,max} \cdot D_{M}^{2}} \right)$$
(2.13)

<u>Step 20</u>: Verify design displacements and forces against code minimum values. Also verify reported base shears against code minimum values.

<u>Step 21</u>: Verification of performance as suggested by the prototype bearing test results. Upon the availability of prototype bearing test results, revise the mathematical model constructed in step 14 to reflect the lower bound and upper bound bearing properties suggested by the prototype test results.

<u>Step 22</u>: Verification of performance as suggested by the production bearing test results. Upon the availability of production bearing test results, revise the mathematical model constructed in step 14 to reflect the lower bound and upper bound bearing properties suggested by the production test results and actual distribution of individual isolators.

2.4 IMPLEMENTATION OF THE DESIGN OF ISOLATED STRUCTURES

Step 1: Given dimension of columns and beams of the superstructure. The value given related to the isolation system is the effective damping β_{eff} (LRB 20%, HDNR 10%, FEMA) and the nondimensional characteristic strength a related to the mechanical characteristics of the dampers.

2.4.1 Preliminary Design Steps

Step 2: First assumption of the desired isolated periods of vibration T_D and T_M .

$$T_D = 3 \cdot T_1$$

$$T_M \ge 3.0 \text{ sec}$$
(2.14)

Step 3: Calculation of the target values of effective stiffness K_{D,max}, K_{D,min}, K_{M,max}, K_{M,min}

$$K_{D,\min} = \frac{(2\pi)^2 \cdot W}{T_D^2}$$
, $K_{D,\max} = 1.10 \frac{K_{D,\min}}{0.90}$ (2.15)

$$K_{M,\min} = \frac{(2\pi)^2 \cdot W}{T_M^2}$$
, $K_{M,\max} = 1.10 \frac{K_{M,\min}}{0.90}$ (2.16)

Step 4: Calculation of the initial estimation of the minimum lateral displacement D_D and D_M .

$$D_{D} = \frac{(g / 4\pi^{2}) \cdot Sa_{D}(T_{D})}{\beta_{D}}, \quad D_{M} = \frac{(g / 4\pi^{2}) \cdot Sa_{M}(T_{M})}{\beta_{M}}$$
(2.17)

$$D'_{D} = \frac{D_{D}}{\sqrt{1 + \left(\frac{T}{T_{D}}\right)^{2}}}, \quad D'_{M} = \frac{D_{M}}{\sqrt{1 + \left(\frac{T}{T_{M}}\right)^{2}}}$$
(2.18)

Where $Sa_D(T_D)$, $Sa_M(T_M)$ are values from the response spectrum 10% in 50 years and 2% in 50 years with damping β_D and β_M , respectively.

Step 5: Check if the calculated D_D is larger than the one selected from the designer D_{sel} . If yes then the T_D in step 2 is getting lower value and the procedure is repeated from step 2.

Step 6: Calculation of the minimum lateral design forces of the superstructure V_s and the isolation system V_b .

$$V_{S} = \frac{N_{iso} \cdot K_{D,max} \cdot D_{D}}{R_{1}}, R_{1} = 2.0, N_{iso}: number of isolators$$
(2.19)

$$V_b = N_{iso} \cdot K_{D,max} \cdot D_D \tag{2.20}$$

Step 7: Perform a linear elastic analysis with triangular distribution of V_s with reference to maximum drift < 0.010/R₁ and stress limits.

Step 8: Check if the value of V_s in step 6 is close to the value calculated from the structure designed in step 7, as shown in the following Eq. 2.21

$$V_S \approx V_{codeshear} = m_{tot} \cdot Sa(T_1) \tag{2.21}$$

where $Sa(T_1)$ from the response spectrum of the fixed superstructure. If yes continue to step 9 and if not penalize the design.

Step 9: Define the mechanical characteristics of the isolators.

2.4.2 Final Design Steps

Step 10: Define FE model superstructure + isolation system.

Step 11: Select analysis design procedure (NSP in this work). The target displacement for the NSP are D'_{D} and D'_{M} calculated previously in step 4.

Step 12: Iteratively define final design satisfying the following performance objectives.

PBD fixed			PBD isolated	
50% / 50y	$\theta_{\rm max} < 0.4\%$	10% / 50 y	$\theta_{\rm max} < 0.4\%$	in displacement D'_D
10% / 50y	$\theta_{\rm max}$ <1.8%	2%/50y	$\theta_{\rm max}$ <1.8%	in displacement $D'_{\scriptscriptstyle M}$
2%/50y	$\theta_{\rm max} < 3.0\%$			

If the previous checks are not fulfilled, penalize the design.

Step 13: Buckling check of each isolator $SF \ge SF_{target}$

$$SF = \frac{\sqrt{2} \cdot \pi \cdot S \cdot \omega_{\rm H} \cdot r}{g} \qquad (2.22)$$

$$\omega_{\rm H} = \frac{K_{\rm H}}{W} \cdot g \quad , \quad K_{\rm H} = \frac{G \cdot A_{\rm s}}{h} \quad , \quad A_{\rm s} = A \cdot \frac{h}{t_{\rm r}} \quad ,$$

where S is the shape factor of the bearing,

 ω_H is the horizontal frequency,

r is the radius of gyration and is equal to $\alpha/2\sqrt{3}$ for a square bearing with side dimension α and $\Phi/4$ for a circular bearing with diameter Φ ,

 K_H is the horizontal stiffness of the bearing,

W is the load carried by the bearing,

A_s is the effective shear area of the bearing,

h is the total height of the bearing (rubber plus steel) and

 t_r is the total height of the rubber.

If the previous check is not fulfilled penalize the design.

Step 14: Lateral displacement check of each isolator

square
$$P_{crit} = B \left[1 - \left(\frac{P}{P_{crit}} \right)^2 \right], \qquad P_{crit} = \frac{\pi}{2\sqrt{2}} \cdot S \cdot S_2 \cdot G$$
 (2.23)

circular
$$P_{crit} = 2R \cdot \frac{\pi}{4} \left[1 - \left(\frac{P}{P_{crit}} \right)^2 \right], \quad P_{crit} = \frac{\pi}{\sqrt{6}} \cdot S \cdot S_2 \cdot G$$
 (2.24)

where B is the side dimension for a square bearing,

R is the radius of the circular bearing,

P is the specified load,

P_{crit} is the critical stress,

S is the shape factor,

 S_2 is the aspect ratio or the second shape factor defined by $S_2 = \Phi/t_r$ or α/t_r and

G is the shear stiffness.

If the previous check is not fulfilled penalize the design.

In figures 2.2 and 2.3 the procedures described are illustrated.

DESIGN OF ISOLATED STRUCTURES



Figure 2.2 Preliminary design procedure



Figure 2.3 Final design procedure

2.5 BUCKLING AND STABILITY OF ELASTOMERIC ISOLATORS

A multilayered elastomeric bearing can be susceptible to a buckling type of instability similar to that of an ordinary column but dominated by the low-shear stiffness of a bearing. The buckling analysis treats the bearing as a continuous composite system. This analysis considers the bearing to be a beam and the deformation is assumed to be such that plane sections normal to the undeformed central axis remain plane, but not necessarily normal to the deformed axis.

To model the rubber isolator as a continuous beam, it is necessary to introduce certain modifications to the defined quantities. Consider the bearing to be a column of length h with a cross-sectional area A and define the shear stiffness per unit length as $P_s = GA_s$, where A_s is an effective shear area given by

$$A_s = A \frac{h}{t_r} \tag{2.25}$$

where *h* is the total height of the bearing (rubber plus steel) and t_r is, as defined earlier, the total height of the rubber. The increase in *A* is needed to account for the fact that steel does not deform in the composite system. The bending stiffness is similarly modified, so that $(EI)_{eff}$ for a single pad of thickness *t* becomes EI_s , where

$$EI_s = E_{c\left(\frac{1}{3}\right)} I \frac{h}{t_r}$$
(2.26)

In terms of these quantities, the overall horizontal stiffness K_H (which was GA/t_r) becomes:

$$K_H = \frac{GA_s}{h} \tag{2.27}$$

and the Euler buckling load for a column with no shear deformation is:

$$P_E = \pi \frac{EI_s}{h^2} \tag{2.28}$$

The bearing is constrained against rotation at both ends and is free to move sideways at the top. The result for the critical buckling load P_{crit} is the solution of the equation:

$$P_{crit} = \frac{-P_s + \sqrt{P_s^2 + 4P_s P_E}}{2}$$
(2.29)

If we assume that $P_s \approx GA$ and $P_E \approx \frac{1}{3} \frac{6GS^2 I \pi^2}{h^2} \approx GA\left(\frac{2\pi^2 S^2 I}{A}\right)$ then, for most types of bearings where $S \ge 5$, $P_E \gg P_S$, the critical load can be approximated by:

$$P_{crit} = (P_S P_E)^{1/2}$$
(2.30)

The usual situation for a bearing in an isolation system is shown in Fig. 2.4. The bearing buckles with no lateral-force constraint but is prevented from rotating at each end.

Using this expression and recalling that:

$$P_{S} = GA \frac{h}{t_{r}} \qquad P_{E} = \frac{\pi^{2}}{h^{2}} \frac{1}{3} E_{c} I \frac{h}{t_{r}}$$
(2.31)

we have

$$P_{crit} = \begin{cases} \left(GA\frac{h}{t_r}\right)^{1/2} \left(\frac{\pi^2}{h^2}\frac{1}{3}6GS^2Ar^2\frac{h}{t_r}\right)^{1/2} \\ \frac{\sqrt{2}\pi GASr}{t_r} \end{cases}$$

where the radius of gyration is denoted by $r = \sqrt{I/A} = \alpha/2\sqrt{3}$ for a square bearing with side dimension α and $\Phi/4$ for a circular bearing with diameter Φ .



Figure 2.4 Boundary conditions for an isolation bearing under a vertical load P

The critical pressure $p_{crit}=P_{crit}/A$ can be expressed on terms of S and the quantity S_2 , referred to as the aspect ratio or the second shape factor, defined by:

$$S_2 = \frac{\Phi}{t_r} \text{ or } \frac{\alpha}{t_r}$$

thus

$$\frac{p_{crit}}{G} = \begin{cases} \frac{\pi}{2\sqrt{2}}SS_2 & \text{for a circular bearing} \\ \frac{\pi}{\sqrt{6}}SS_2 & \text{for a square bearing} \end{cases}$$
(2.32)

In actual design, the load *W* carried by a bearing will be less than the critical load and neglecting the effect of the vertical load on the horizontal stiffness K_H of the bearing, that is given by $K_H = GA/t_r$, which in turn is related to the horizontal frequency ω_H , through:

$$\omega_H^2 = \frac{K_H}{W}g$$

Thus, the safety factor SF against buckling, which is defined by $SF=P_{crit}/W$. becomes:

$$SF = \frac{\sqrt{2\pi}S\omega_H^2 r}{g} \tag{2.33}$$

All other things being equal, the safety factor increases with shape factor *S*, frequency ω_H , or bearing size (either α or Φ).

The bearing size depends on the carried load. If the pressure p=W/A is specified, then:

$$r = \begin{cases} 2\sqrt{\pi} \left(\frac{W}{p}\right)^{1/2} & \text{for a circular bearing} \\ 1 & 1/2 \end{cases}$$
(2.34)

$$\frac{1}{\sqrt{2}} \left(\frac{W}{n}\right)^{1/2}$$
 for a square bearing (2.35)

If the pressure is fixed, the safety factor will diminish as $W^{1/2}$, leading to the unexpected result that buckling can become a problem for bearings that are lightly loaded.

2.5.1 Influence of Vertical Load on Horizontal Stiffness

When the load carried by the bearing is comparable to the buckling load, the horizontal stiffness K_H is reduced. The reduction is obtained by using the same linear elastic analysis and is given by:

$$K_H = \frac{GA_S}{h} \left[1 - \left(\frac{P}{P_{crit}}\right)^2 \right]$$
(2.36)

If the load is less than 0.32 times the buckling load, the accuracy of the usual formula for K_H is better than 10% and in most designs that requirement will ensure that this is the case.

The downward displacement δ_V of the top of a bearing carrying a vertical load P and displaced through a sideways movement at the top of D is also given by the buckling analysis in the form:

$$\delta_V = \frac{P_S + P}{P_E} \frac{D^2}{h} \tag{2.37}$$

In most cases, $P \gg P_S$; thus:

$$\delta_V = \frac{P}{P_{crit}} \frac{P_{crit}}{P_E} \frac{D^2}{h} = \left(\frac{P}{P_{crit}}\right) \sqrt{\frac{P_S}{P_E}} \frac{D^2}{h}$$
(2.38)

Now,

$$\frac{P_S}{P_E} = \frac{GA_S h^2}{2\pi^2 IS^2}$$
(2.39)

In terms of $r = (I/A)^{1/2}$, we have:

$$\frac{\delta_V}{h} = \frac{P}{P_{crit}} \frac{h}{\sqrt{2\pi}rS} \frac{D^2}{h^2}$$
(2.40)

This downward displacement is in addition to that produced by pure compression of the isolator and is caused by the rotation of the reinforcing steel shims in the center of the

bearing. This rotation produces a shear stress caused by the component of the vertical load along the rotated layers and the resulting shear strain causes the downward movement of the top of the bearing.

2.5.2 Stability under Large Lateral Displacement

The buckling analysis for an elastomeric isolator is based on the linear theory that is analogous to the buckling analysis of a column and, as is the case in the usual theory, provides the buckling load or buckling stress in the undisplaced position but no information on the stability of a bearing in the displaced position, the instability will manifest itself by the loss of positive incremental horizontal stiffness. This type of instability is of crucial importance in bearing design since the peak downward load on an isolator will occur at the same time as the peak horizontal displacement and in combination will be one of the limit states for which the isolator will need to be proportioned.

In principle, a complex nonlinear analysis will be needed to predict the bearing behavior under the combination of peak vertical load and maximum horizontal displacement. There are two simple hypotheses for an approximation to the limit state when an isolator is loaded in shear and with vertical load.

The first is that the critical displacement, defined as the displacement under which that bearing exhibits zero incremental horizontal stiffness, is the lateral displacement at which the reduced area compression stress calculated from the axial load divided by A_r (the area of overlap between top and bottom) reaches the critical stress p_{crit} given by Eq. (2.32).

The second hypothesis is that the area A in the expression for the critical load in the underformed configuration [Eq. (2.31)] is replaced by the reduced area A_r . This is the most plausible for the two possibilities as the concentration of the vertical stress due to displacement will not affect the bending resistance but could reduce the resistance due to shear.

For a square bearing of side dimension B, the reduced area A_r is given by:

$$A_r = B(B - D) \tag{2.41}$$

so that if the first hypothesis is correct, the critical displacement D_{crit} under a specified load *P* is given by:

$$P = p_{crit}A_r = \frac{\pi}{\sqrt{6}}GSS_2B(B - D_{crit})$$
(2.42)

Therefore,

$$\frac{D_{crit}}{B} = 1 - \frac{P}{P_{crit}}$$
(2.43)

On the other hand, if the second hypothesis is correct, the critical displacement is given by:

$$P = \sqrt{GA_r \frac{\pi^2 (EI)_{eff}}{t_r^2}} = \left(\frac{A_r}{A}\right)^{1/2} P_{crit}$$
(2.44)

$$\frac{A_r}{A} = \left(\frac{P}{P_{crit}}\right)^2 = \frac{B(B - D_{crit})}{B^2}$$

$$1 - \frac{B}{D_{crit}} = \left(\frac{P}{P_{crit}}\right)^2 \quad or \quad \frac{D_{crit}}{B} = 1 - \left(\frac{P}{P_{crit}}\right)^2 \tag{2.45}$$

Both results are the same for *P* close to P_{crit} but differ for the range of practical application where $P < P_{crit}$.

The overlap area for a square bearing is easy to calculate, while for the circular bearing of radius R it is harder.

2.5.3 Rollout Stability

An isolation bearing, even if inherently stable under its design load, can experience another form of instability if it is connected to the foundation below and the superstructure above through shear keys that cannot sustain tensile loads. Initially designers felt that rubber should not be subjected to tension. Therefore, early designs of rubber bearings used dowelled shear connections rather than bolted connections. Dowelled bearings, however, can experience an unstable mode of behavior, called "rollout", which is associated with lateral displacement and puts a limit on the maximum displacement that the bearing can sustain. The bearing is unstable in the sense that beyond this displacement the force-displacement curve has a decreasing slope. Because the bearing cannot sustain tension, the movement at the top and bottom of the bearings is produced by a change in the line of action of the resultant of the vertical load, as shown in Fig. 2.5a. The limit of this migration of the resultant is reached when the resultant is at the edge of the bearing and the equilibrium of the moment generated by the lateral force F_H with that generated by the vertical load P gives:

$$P(b - \delta_{max}) = hF_H \tag{2.46}$$

Where *b* is the bearing width (either α if square of Φ if circular). The relationship between the lateral force F_H and the displacement δ is shown if Fig. 2.5b.



Figure 2.5 Mechanics of rollout for dowelled bearings

If we take K_H as GA/t_t and the pressure p=P/A, this becomes:

$$\frac{\delta_{max}}{b} = \frac{1}{1 + (G/p)(h/t_r)}$$
(2.47)

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CHAPTER 3

ANALYSIS PROCEDURES AND DESIGN

CHAPTER 3

3.1 INTRODUCTION

Seismic analysis is the calculation of the response of a building (or nonbuilding) structure to earthquakes. It is a part of the process of structural design or structural assessment and retrofit in regions where earthquakes are prevalent. In order to assess the behavior of a structure against a seismic hazard, different analysis procedures are used. According to FEMA-356 (2000) four alternative analytical procedures, based on linear and nonlinear structural response, are recommended for the structural analysis of buildings under earthquake loading.

In this study, three analysis procedures are examined: The Linear Static Procedure (LSD), the Nonlinear Static Procedure (NSP) and the Nonlinear Dynamic Procedure (NDP).

3.2 LINEAR STATIC PROCEDURE (LSP)

Most structural analysis problems can be treated as linear static problems, based on the following assumptions:

- *Small deformations*: Loading pattern is not changed due to the deformed shape.
- *Elastic materials*: No plasticity or failures.
- *Static loads*: The load is applied to the structure in a slow or steady fashion.

Linear analysis can provide most of the information about the behavior of a structure and can be a good approximation for many analyses. It is also the bases of nonlinear analysis in most of the cases. However, when a linear analysis method is employed, simplifying assumptions of the structural response are made, resulting either to conservative and therefore to more expensive designs, or to designs with reduced safety, since phenomena that have not been taken into account during the design phase may influence the load carrying capacity of the structure during its functioning.

3.3 NONLINEAR STATIC PROCEDURE (NSP)

NSP, also called Pushover Analysis, is recommended by FEMA-356. It is the most commonly used analysis procedure, in order to be calculated the capacity of a specific structure. This method is approximated due to the fact that seismic hazards as a dynamic phenomenon simulated by static loads.

The purpose of this procedure is:

- The assessment of the structural performance in terms of strength and deformation capacity globally, as well as at the element level.
- The identification of the regions of high inelastic deformations.
- Obtaining the sequence of the member yielding and the creation of plastic hinges.

The structural model is "pushed" according to a predefined lateral load pattern. The whole procedure is based on the assumption that the response is related to the response of an equivalent single degree of freedom system. The lateral loads are imposed in conjunction with the imposition of gravity loads. The structure is "pushed" under lateral loads, which are increased proportionally, until the target displacement is reached from a characteristic node of the structure model, or earlier if the algorithm fails to converge because a collapse mechanism has been formed.

3.3.1 Lateral Loads

According to FEMA-356 lateral loads shall be applied to the mathematical model in proportion to the distribution of inertia forces in the plane of each floor diaphragm. For all analyses, at least two vertical distributions of lateral load shall be applied. One pattern shall be selected from each of the following two groups:

- 1. The first distribution should be one of the following three:
 - A vertical distribution proportional to the shape of the fundamental mode in the direction under consideration:

$$F_i = V_b \frac{\varphi_i \cdot W_i}{\sum_{j=1}^N \varphi_j \cdot W_j}$$
(3.1)

where:

V_b is the base shear,

 φ_i, φ_j are the i^{th} and the j^{th} eigenmode, respectively and

W_i, W_j are the portion of the total building weight W located on or assigned to the floor level and j, respectively.

The use of this distribution shall be permitted only when more than 75% of the total mass participates in this mode.

•
$$F_i = V_b \frac{h_i^k \cdot W_i}{\sum_{j=1}^N h_j^k \cdot W_j}, k = \begin{cases} 1.0, & T \le 0.5 \text{ sec} & (FEMA - 356) \\ 1.0 + 0.5(T - 0.5), & 0.5sec < T < 2.5 \text{ sec}(FEMA - 356) \\ 2.0, & T \ge 2.5 \text{ sec} & (FEMA - 356) \\ 1.0 \text{ in any case} & (EC8) \end{cases}$$

where:

V_b is the base shear,

 h_i , h_j is the height from the base to floor level I and j, respectively and

 W_i , W_j are the portion of the total building weight W located on or assigned to floor level I and j, respectively.

The use of this distribution shall be permitted only when more than 75% of the total mass participates in the fundamental mode in the direction under consideration and the uniform distribution is also used. For k=0 the distribution is uniform, for k=1 is inversed triangular and k=2 is parabolic.

- A vertical distribution proportional to the story shear distribution calculated by combining modal responses from a response spectrum analysis of the building, including sufficient modes to capture at least 90% of the total building mass and using the appropriate ground motion spectrum. This distribution shall be used when the period of the fundamental mode exceeds 1.0 second.
- 2. The second pattern should be chosen between the two following recommended distributions:
 - A uniform distribution consisting of lateral forces at each level proportional to the total mass at each level.
 - An adaptive load distribution that changes as the structure is displaced. The adaptive load distribution shall be modified from the original load distribution using a procedure that considers the properties of the yielded structure.

The purpose being taken into account two types of distributions is the definition of the structure's response more accurately. Seeing that the simulated lateral loads according to the uniform distribution are more similar to the real loads in case the structure sustains considerable damage, the combination of the two at least modes in the derivation of the imposed lateral loads is in demand.

3.3.2 Target Displacement

The target displacement should be calculated in order to determine if the capacity is higher or lower than the demand. The target displacement is the displacement of the characteristic node when the structure is under the design loads. The value of the target displacement can be approached through the following methods, the second of which is the one applied in this study (FEMA-356):

- Capacity spectrum method,
- Displacement coefficient method,
- Equal coefficient method.

3.3.3 Displacement coefficient method

The target displacement δ_t at each floor level, is calculated in accordance with Eq. 3.2:

$$\delta_t = C_0 \cdot C_1 \cdot C_2 \cdot C_3 \cdot S_\alpha \cdot \frac{T_e^2}{4\pi^2} g$$
(3.2)

where:

 C_0 is the modification factor to relate spectral displacement of an equivalent SDOF system to the roof displacement of the building MDOF system calculated using one of the following procedures:

- The first modal participation factor at the level of the control node,
- The modal participation factor at the level of the control node calculated using a shape vector corresponding to the deflected shape of the building at the target displacement.
- The appropriate value from Table 3.1 (Table 3-2 of FEMA-356).

	Other buildings		
Number of Stories	Triangular Load Pattern	Uniform Load Pattern	Any Load Pattern
1	1.0	1.0	1.0
2	1.2	1.15	1.2
3	1.2	1.2	1.3
5	1.3	1.2	1.4
10+	1.3	1.2	1.5

Table 3.1	Values	for	Modification	Factor	$C_{a}(1)$)
10010 3.1	vulues	101	Widdiffcution	i uctor	C0 (1	-)

Linear interpolation shall be used to calculate intermediate values.

(1): Buildings in which, for all stories, interstory drift decreases with increasing height.

 C_1 is the modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response:

$$C_{1} = \begin{cases} 1.0 & for \ T_{e} \ge T_{s} \\ \frac{\left[1.0 + \frac{(R-1)T_{s}}{T_{e}}\right]}{R} & for \ T_{e} < T_{s} \end{cases}$$
(3.3)

 T_e is the effective fundamental period of the building in the direction under consideration, sec.

 T_s is the characteristic period of the response spectrum, defined as the period associated with the transition from the constant acceleration segment of the spectrum.

R is the ratio of elastic strength demand to calculated yield strength coefficient, calculated by Eq. 3.4.

 C_2 is the modification factor to represent the effect of pinched hysteretic shape, stiffness degradation and strength deterioration on maximum displacement response. Values of C_2 for different framing systems and Structural Performance Levels shall be obtained from Table 3.2 (Table 3-3 of FEMA-356). Alternatively, use of C_2 =1.0 shall be permitted for nonlinear procedures.

	<i>T</i> ≤0.1 se	cond (3)	<i>T</i> ≥ <i>Ts</i> second (3)		
Structural Performance Level	Framing Type 1 (1)	Framing Type 2 (2)	Framing Type 1 (1)	Framing Type 2 (2)	
Immediate Occupancy	1.0	1.0	1.0	1.0	
Life Safety	1.3	1.0	1.1	1.0	
Collapse Prevention	1.5	1.0	1.2	1.0	

Table 3.2 Values for Modification Factor C₂

(1): Structures in which more than 30% of the story shear at any level is resisted by any combination of the following components, elements or frames: ordinary moment-resisting frames, concentrically-braced frames, frames with partially-restrained connections, tension-only braces, unreinforced masonry walls, shear-critical, piers and spandrels of reinforced concrete or masonry.

(2): All frames not assigned to Framing Type 1.

(3): Linear interpolation shall be used for intermediate values of T.

 C_3 is the modification factor to represent increased displacements due to dynamic P- Δ effects. For buildings with positive post-yield stiffness, values of C_3 shall be set equal to 1.0. For buildings with negative post-yield stiffness, values of C_3 shall be calculated using Eq. 3.5.

 S_{α} is the response spectrum acceleration, at the effective fundamental period and damping ratio of the building in the direction under consideration.

g is the acceleration of gravity.

The strength ratio *R* is calculated in accordance with the equation:

$$R = \frac{S_{\alpha}}{(V_y/W)} \cdot C_m \tag{3.4}$$

where S_{α} is defined above and:

 V_y is the base shear when the yield strength of the structure is calculated using results of the NSP.

W is the effected seismic weight.

m is the effective mass factor from Table 3.3 (Table 3-1 of FEMA-356).

$$C_3 = 1.0 + \frac{|\alpha|(R-1)^{3/2}}{T_e}$$
(3.5)

where R and T_e are defined above and

 α is the ratio of post-yield stiffness to effective elastic stiffness, where the nonlinear force-displacement relation shall be characterized by a bilinear relation.

No. of Stories	Concrete Moment Frame	Concrete Shear Wall	Concrete Pier- Spandrel	Steel Moment Frame	Steel Concentric Braced Frame	Steel Eccentric Braced Frame	Other
1-2	1.0	1.0	1.0	1.0	1.0	1.0	1.0
3 or							
more	0.9	0.8	0.8	0.9	0.9	0.9	1.0
<i>Cm</i> shall be taken as 1.0 if the fundamental period, <i>T</i> , is greater than 1.0 second.							

Table 3.3 Values for Effective Mass Factor Cm

3.4 NONLINEAR DYNAMIC PROCEDURE (NDP)

The principal concept to be dealt with is that of a dynamical system. A dynamical system is a model that determines the evolution of a system, given only the initial state, which implies that these systems posses memory: the current state is a particular function of a previous state. Thus a dynamical system is described by two things: a state and a dynamics. The state of a dynamical system is determined by the values of all the variables that describe the system at a particular moment in time. The dynamics of the system is the set of laws or equations that describe how the state of the system changes over time.

In the seismic assessment of structures a wide range of seismic records and more than one performance levels should be considered in order to take into account the uncertainties that the seismic hazard introduces into a performance-based seismic assessment or design problem. The methods used for the performance-based assessment implementing nonlinear dynamic analyses are classified as single and multiple hazard level methods. IDA and MSDA are the two lost applicable methods, both considering multiple hazard levels.

3.4.1 Incremental Dynamic Analysis (IDA)

The main object of an IDA study is to develop a curve through a relation between the seismic intensity level and the corresponding maximum response of the structural system. The intensity level and the structural response are described through IN and EDP, respectively.

The IDA study is implemented through the following steps:

- 1. Defining the nonlinear FE model required for performing nonlinear dynamic analyses;
- 2. Selecting a suit of natural records;
- 3. Selecting a proper intensity measure and an engineering demand parameter;

- 4. Employing an appropriate algorithm for selecting the record scaling factor, in order to obtain the IDA curve performing the least required nonlinear dynamic analyses and
- 5. Employing a summarization technique for exploiting the multiple records results.

3.4.2 Multicomponent Incremental Dynamic Analysis (MIDA)

In order to take into account damage and other earthquake losses, a reliable tool for estimating the capacity for any structural system in multiple earthquake hazards is required. MIDA, proposed by Lagaros (2010), is performed in a similar way of the 2D implementation of IDA, but in 3D way. A suit of records is selected and for each record a MIDA representative curve is derived. The 50% fractile MIDA curve is then calculated using the representative curves of all the records.

3.4.3 Multiple-Stripe Dynamic Analysis (MSDA)

MSDA is a nonlinear dynamic analysis method that can be used for performance-based seismic assessment of structures for a wide range of ground motions and more than one performance levels. Similar to IDA, the main objective of a MSDA study is to define a relation between the seismic intensity level and the corresponding maximum response of the structural system. The intensity level and the structural response are described through an IM and an EDP, respectively. The method refers to groups of inelastic dynamic analyses (stripes) performed at multiple spectral acceleration levels, where at each stripe analysis a number of dynamic structural analyses are performed for a group of ground motion records that are scaled to a single value of spectral acceleration. It is common to use the same suite of records for all the spectral acceleration levels.

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CHAPTER 4

STRUCTURAL DESIGN OPTIMIZATION

CHAPTER 4

4.1 INTRODUCTION

It is fairly accepted that one of the most important human activities is decision making, no matter where the field of activity belongs to. Optimization techniques play an important role in structural design, the very purpose of which is to find the best solutions from which a designer or a decision maker can derive a maximum benefit from the available resources.

In order to avoid any misunderstanding, it is important to define the term *structure*. The term *structure* is used to describe the arrangement of the elements and/or the materials, in order to create a system capable to undertake the loads imposed by the design requirements. It is crucial that the structural system is optimally designed. The term *optimum design* is used for a design that satisfies both the serviceability requirements and other important criteria like the cost or the weight of the system.

The basic idea behind intuitive or indirect design in structures is the memory of past experiences, subconscious motives or random selections. This, in general, will not lead to the best design. The shortcomings of the indirect design can be overcome by adopting a direct or optimal design procedure. The feature of the optimal design is that it consists of only logical parameters. In making a logical decision, one sets out the objective function, which could be either cost or weight function.

Structural optimum design methods can also be according to the design philosophy employed. Most civil engineering structures are even today designed on the basis of permissible stress criterion. However, some of the recent methods use a specified factor of safety against ultimate failure of the structure. Presently, the approach is based on the design constraints expressing the maximum probability of various types of events such as local or ultimate failure. The objective function is obtained by calculating each event and multiplies it by the corresponding probability. The sum of all such products will be the total objective function. The constraints may also be probabilistic. These are suitable in situations when the loads acting on the structure are probabilistic or the material properties are random.

The aim of the engineer is to find a combination of independent variable, called *parameters* or *design variables*, so as to optimize the objective function of the problem. In general the design variables are real but sometimes they could be integers, for example, number of layers, orientation angle, etc. The behavior constraints could be equality constraints or inequality constraints depending on the nature of the problem. If the objective function and the constraints involving the design variable are linear then the optimization is termed as linear optimization problem. If even one of them is non-linear it is classified as the non-linear optimization problem.

In order to calculate the optimal designs, it is necessary to perform two steps: the mathematical formulation of the optimization problem and the execution of an

optimization algorithm. The first step is to define the parameters and the relationship between them, to determine the optimization function and to define the constraints of the problem. The optimization process is completed by choosing a suitable optimization algorithm and its combination with the structural and the optimization models. A basic requirement for the case of structural optimal design is to express in mathematical terms the structural behavior. In the case of structural systems behavior this refers to the response under static and dynamic loads, such as displacements, stresses, eigenvalues, buckling loads, etc. The optimum design procedure for structural systems is presented in Figure 4.1.



Figure 4.1 The structural optimization procedure

4.2 HISTORY OF OPTIMIZATION

Optimization problems made their appearance in ancient Greece, when Greek mathematicians solved some optimization problems related to their geometrical studies. In 300 B.C., Euclid considered the minimum distance between a point and a line and proved that a square has the greatest area among the rectangles with given total length of edges, while in 100 B.C., Herod proved in *Catoprica* that light travels between two points through the path with shortest length when reflecting from a mirror.

Before the invention of calculus of variations, only some separate optimization problems were investigated. In 1646 P. de Fermat showed that at the extreme point the gradient of a function vanishes and developed the more general principle that light travels between two points in a minimum time. I. Newton (1660s) and G. W. von Leibniz (1670s) created mathematical analysis that forms the basis of calculus of variations (CoV), while some separate finite optimization problems were also considered.

During the 19th century the first optimization algorithms were presented. In 1806 A.-M. Legendre presented the least square method, which also J.C.F. Gauss claimed to have

invented, while Cauchy in 1847 presented for the first time a minimization procedure (Steepest Descent Method) implementing function derivatives. In 1857 J.W. Gibbs showed that chemical equilibrium is an energy minimum.

In 1917 H. Hancock published the first book on optimization, *Theory of Minima and Maxima*, while biomathematician D.W. Thompson wrote the book *On Growth and Form*, in which he applied optimization to analyze the forms of living organisms. In 1928 F.P.P. Ramsey applied CoV in his study on optimal economic growth. G. Dantzig (1947) presented the Simplex method for solving LP-problems, while W. Karush (1939), F. John (1948), as well as H.W. Kuhn and A.W. Tucker (1951) invented optimality conditions for nonlinear problems, initiating the modern era of optimization. In 1957, R. Bellman presented the optimality principle.

During the 60s, several optimization methods for solving nonlinear problems were introduced. Zoutendijk (1960), Rosen, Wolfe and Powell presented the methods of feasible directions to generalize the Simplex for nonlinear programs, Rosenbrock (1960) presented the method of orthogonal directions, Hooke and Jeeves (1961) developed the pattern search method, Davidon, Fletcher and Powell (1963) stated the variable metric method, Fletcher and Reeves (1964) presented the Conjugate Gradient method, Powell (1964) introduced the method of conjugate directions, Nelder and Mead (1965) suggested their Simplex method, Box (1965) introduced his homonymous technique, while Fiacco and McCormick (1966) formed the so called Sequential Unconstrained Minimization technique.

In 1973 Mathematical Programming Society was founded. Since 1970 structural optimization was the subject of intensive research and several different approaches for optimal design of structures had been advocated. All the aforementioned methods were of deterministic character, because of the fact that that the element of the randomness was non-existent. That means, when applied to the same initial design vector, they always result to the same final design vector.

In contrast to the deterministic optimization methods, the stochastic optimization procedures allow for randomness to appear. In this way, it is possible to get different final design vectors, even though the initial vector is the same. Apart from the pure deterministic or pure stochastic procedure, hybrid schemes have been introduced as well. The main idea behind hybridism is to combine the advantages of both categories of methods for a better result to be obtained.

4.3 FORMULATION OF THE STRUCTURAL OPTIMIZATION PROBLEM

A structural optimization problem may be *continuous* or *discrete*, depending on the type of the design space that the design parameters take values. Due to code requirements

often the range of the design space is discrete. The mathematical model of a continuous optimal design problem can be formulated as follows:

$$F(\mathbf{s}) \to \min$$

$$\mathbf{s} = \{s_1, s_2, ..., s_n\}^T$$

$$s_i^L \le s_1 \le s_i^U, i=1,2,...,n$$

$$g_j(\mathbf{s}) \ge 0, j=1,2,...,m$$

$$h_j(\mathbf{s}) = 0, j=m+1, m+2,...,t$$

(4.1)

where **s** is the vector of design variables, s_i^L and s_i^U are the lower and upper bound of design variable s_i , respectively, $F(\mathbf{s})$ is the objective function to be minimized, while $g_j(\mathbf{s})$, $h_j(\mathbf{s})$ are the inequality and equality constraint functions, respectively.

Many of the parameters of the problems of structural design optimization may be discrete or mixed. A typical optimal design problem with mixed parameters is the shapesizing design optimization problem of a truss structure. In such a problem, the coordinates of the nodes of the panel, which determine the optimum shape of the structural system, can take continuous values, however standardization reasons require sectional bars to be defined from a discrete design space. Such problem is called as a mixed-discrete design optimization problem.

In accordance to the problem described in Eq. (4.1), a discrete optimal design problem can be written as follows:

$$F(\mathbf{s}) \to \min$$

$$\mathbf{s} = \{s_1, s_2, ..., s_n\}^T$$

$$s_i^L \le s_1 \le s_i^U, i=1,2,...,n$$

$$s_i \in \mathbb{R}^d, i=1,2,...,n$$

$$g_j(\mathbf{s}) \ge 0, j=1,2,...,m$$

$$h_j(\mathbf{s}) = 0, j=m+1,m+2,...,t$$

(4.2)

where R^d is the design space of the discrete design variables **s**. The design variables s_i (i=1,2,...,n) can take values only from the design set R^d .

Usually in the case of a mixed-discrete or a purely discrete problem, the design variables are dealt as though they were continuous design variables; while at the end of the process, once the optimal values of all design variables have been determined, appropriate values derived from the discrete design space are assigned to the continuously defined design variables (Hager and Balling, 1988).
4.3.1 Design Variables

If the values of the parameters obtained to the design are fully defined, the parameters are called *design variables*. If a design does not fulfill the design requirements of the problem, it is called *infeasible*; otherwise it is called *feasible design*. One feasible design is not necessarily the best, but it is always able to be implemented. The first and most important step for proper formulation of a problem is to choose the correct design variables. If the selection of the design obtained from the optimization algorithm is infeasible. For this reason, in some cases it is desirable to select more design variables than necessary, although this way the degrees of freedom of the formulation of the optimization problem of the system are increased. Another important issue that needs to be taken into account in the selection of design variables is their relative independence.

It is recommended to conduct a sensitivity analysis, in order to estimate the sensitivity of the objective function over all the design variables, before the final choice of the optimization model. Through the sensitivity analysis it is possible to detect design variables that have negligible influence on the objective function.

4.3.2 Objective Function

Every optimization problem is described by a large number of feasible designs, while the best solution is unique. To make the distinction between good and bad designs, it is necessary to have a mutual criterion for comparing and evaluating the designs. This criterion is defined by a function that takes a specific value for any given design. This function is called as *objective function* which depends on the design variables. The formulations given in Eqs. (4.1) and (4.2) refer to a minimization problem. A maximizing problem of the function F(s) can be transformed into a minimization problem of the objective function that is to be minimized is often called as the *cost function*.

Some examples of objective functions are the minimizing of the cost, the weight optimization problem, the energy losses problem and the maximizing of the profit. In some cases, the formulation of the optimization problem is defined with the simultaneous optimization of two or more objective functions that are conflicting against each other. One example is the case where the objective is to find an optimum design with minimum weight, while having minimum stress or displacement in some parts of the structural system. These type of problems are called *optimization problems with multiple objective functions* (multi-objective design or Pareto optimum design).

4.3.3 Constraint Functions

The design of a structural system is achieved when the design parameters take specific values. All engineering or code provisions are introduced in the mathematical optimization model in the form of inequalities and equalities which are called *constraint functions*. These constraint functions should be at least dependent on one design variable.

The constraint functions that are usually imposed on structural problems are stress and strain constraints, whose values are not allowed to exceed certain limits. However, due to possible uncertainties on the definition of the problem or due to inexperience, sometimes additional constraint functions are imposed that may be useless, which are either dependent on others, or they remain forever in the safe area. The use of these additional constraint functions may result to calculations requiring additional computational effort without any benefit.

One inequality constraint function $g_j(\mathbf{s}) \le 0$ is considered as *active* at the point \mathbf{s}^* in the case that the equality is satisfied, i.e. $g_j(\mathbf{s}^*)=0$. Therefore, the above constraint function is considered as *inactive* for the design \mathbf{s}^* for the case that the inequality is strictly satisfied, i.e. $g_j(\mathbf{s}^*)<0$. If a positive value that $g_j(\mathbf{s}^*)>0$ corresponds to the value of the constraint function, the inequality constraint function is considered to be violated for the design \mathbf{s}^* . Similarly, an equality constraint function $h_j(\mathbf{s})=0$ is considered that it is violated for the design \mathbf{s}^* if the equality is not satisfied, i.e. $h_j(\mathbf{s}^*)\neq 0$. Therefore, an equality constraint function might be active or violated. From all the description provided related to the active or the inactive constraint functions, it is clear that any feasible design is defined by active or inactive inequality constraint functions and active equality constraint functions.

When a constraint function is inactive, it means that its presence is not important at that part of the optimization procedure, since the active constraint functions fulfill the needs of the design. However in another step it may become active. Therefore, at each step of the optimization process, there are both active and inactive constraint functions. It is impossible to determine in advance which of these functions will become active at each step and which of them will remain inactive. For this reason, when solving optimization problems, it is necessary to use different techniques in order to improve the efficiency of the optimization procedure and reduce significantly the time required for the calculations. Consequently, it is crucial to identify at each step of the optimization procedure the constraint functions that are located within the safe area, i.e. they are inactive, which they do not affect the process of finding an improved design, in order to continue the optimization process with only the active constraint functions.

In order to identify the active constraint functions, the values of the constraint functions should be normalized first (Vanderplaats, 1984), to have a single reference

system regardless of the type of the constraint function. For example, if the value of a displacement constraint function takes value about 0.1-1.0 cm, while the value of a stress displacement constraint function takes value about 20,000 kPa, it is necessary to homogenize the sizes of the two constraint functions. The normalization of the value constraint functions takes place by the following relations:

$$g_{j}^{N}(\boldsymbol{s}) = \frac{g_{j}^{L} - g_{j}}{|g_{j}^{L}|} \le 0$$
(4.3)

for a constraint function limited with a *lower bound*, $g_i \ge g_i^L$, and

$$g_{j}^{N}(\boldsymbol{s}) = \frac{g_{j} - g_{j}^{U}}{|g_{j}^{U}|} \le 0$$
(4.4)

for a constraint function limited with an *upper bound*, $g_j \leq g_j^U$. Thereby, if a normalized value of the constraint function is equal to +0.50, it violates its permissible value by 50%, while if its normalized value is equal to -0.50, it is 50% below the allowable value.

It is important to define the *local* and *global minimum* in mathematical terms:

Local minimum: A point \mathbf{s}^* in the design space is considered as a local or a relative minimum if this design satisfies the constraint functions and the relationship $F(\mathbf{s}^*) \leq F(\mathbf{s})$ is valid for every feasible design point in a small region around the point \mathbf{s}^* . If only the inequality is valid, $F(\mathbf{s}^*) < F(\mathbf{s})$, then the point \mathbf{s}^* is called as *strict* or *unique* or *strong local minimum*.

Global minimum: A point s^* in the design space is considered as a global or absolute minimum for the problem at hand if this design satisfies the constraint functions and the relation $F(s^*) \le F(s)$ is valid for every feasible design point. If only the inequality is valid, $F(s^*) < F(s)$, then the point s^* is called as a *strict* or *unique* or *strong global minimum*.

To sum up all of the above, in order to perform an optimization analysis in a structural problem, at first its modeling has to be done, so as to be consisted by the three following basic elements:

- 1. The design variables that determine the design problem;
- 2. The objective function that describes the aim of the optimization;
- 3. The constraint functions that rule the design variables, which guide the research into optimum designs.

4.4 CLASSES OF OPTIMIZATION

There are three main classes of the structural optimization:

• Topology optimization;

- Shape optimization;
- Sizing optimization.

At first, sizing optimization's target is to minimize the weight of the structure, with stress and displacement as constraint functions in its structural members. Thereafter, shape optimization aims to find the optimum boundaries of the structure. Last but not least, through the topology optimization the designer manages to optimize the layout or the topology of the structure by detecting and removing the low-stressed material which is not used effectively in the structure.

4.4.1 Topology optimization

Determining the topology of a structure is very important, so as the design operating conditions are satisfied. After defining the loads undertaken by the structure and the mounting of the structure, by the use of appropriate algorithms, an iterative process is been followed, which leads to the optimum distribution of the material of the structure. After the topology optimization is achieved, then the shape optimization through proper algorithms is attempted so as to achieve finding the optimal structure.



Figure 4.2 Topology optimization for a single loading

4.4.2 Shape optimization

In shape optimization of a structure with a specific topology, the basic target is to improve the performance of the structure by modifying its boundaries. This can be achieved by minimizing the objective function subjected to certain constraints. Specifically, the shape optimization methodology consists of the following steps:

- 1. Defining the geometry of the structure under investigation. The boundaries of the structure are modeled using cubic B-splines and defined by a set of key points. Some of the coordinates of these key points will be the design variables, which may or may not be independent to each other.
- 2. Creating a valid and complete finite element model by using an automatic mesh generator. Thereafter, the displacements and stresses are evaluated through a finite element analysis.

- 3. If a gradient-based optimizer is used then the sensitivities of the constraints and the objective function to small changes of the design variables are computed either with the finite difference, or with the semi-analytical method.
- 4. The optimization problem is solved; the design variables are being optimized and the new shape of the structure is defined. If the convergence criteria for the optimization algorithm are satisfied, then the optimum solution has been found and the process is terminated, else a new geometry is defined and the whole process is repeated from step (2).



Figure 4.3 Shape optimization problem (a) engine block-initial shape and (b) engine block-final shape

4.4.3 Sizing optimization

In sizing optimization problems the aim is usually to minimize the weight of the structure under certain behavioral constraints on stresses and displacements. The design variables are most frequently dimensions of the cross-sectional areas of the members of the structure. The members of the structure are divided into groups having the same design variables. Due to fabrication limitations, the design variables are not continuous but discrete since cross-sections belong to a certain set. A discrete structural optimization problem can be formulated in the following form:

min F(**s**)

subject to g_j(**s**)≤0, j = 1,2,...,m

where R^d is a given set of discrete values representing the available structural member cross-sections and design variables s_i (i = 1,2,...,n) take values only from this set.

(4.5)



Figure 4.4 Sizing optimization problem (a) bridge girder and (b) runway beam

Specifically, the sizing optimization methodology consists of the following steps:

- 1. Defining the geometry, the boundaries and the loads of the structure,
- 2. Selecting the proper design variables, which may or may not be independent to each other and defining the constraints, in order to formulate the optimization problem as in Eq. (4.5).
- 3. Carrying out a finite element analysis and evaluating the displacements and the stresses.
- 4. If a gradient-based optimizer is used then the sensitivities of the constraints and the objective function to small changes of the design variables are computed.
- 5. Optimization of the design variables. If the convergence criteria for the optimization algorithm are satisfied, the optimum solution is found and the process is terminated. Otherwise, the optimizer updates the design variable values and the whole process is repeated from step (3).

4.5 EVOLUTIONARY ALGORITHMS

The two most widely used optimization algorithms belonging to the class of EA that imitate nature by using biological methodologies are the Genetic Algorithms (GA) and the Evolution Strategies (ES).

4.5.1 Genetic Algorithms (GA Method)

The problem of mixed mode variables has been solved by the application of GA. These are suitable for complex optimization problems. Application of genetic algorithms for optimization studies is gaining wide interest because of their robustness in locating the

global optimum. The advantage of GA method is that it manages to optimize composite laminates, since it can handle all types of variables providing the flexibility needed to solve such complex problems.

4.5.2 Evolution Strategies (ES Method)

The ES method is used as the optimization tool for addressing the present problem, based on previous experience regarding the relative superiority of ES over the mathematical programming and GA methods in some specific problems. ES imitate biological evolution in nature and have three characteristics that make them differ from the gradient based optimization algorithms:

- a. In place of the usual deterministic operators, they use randomized operators: recombination, mutation, selection;
- b. Instead of a single design point, they work simultaneously with a population of design points;
- c. They can handle continuous, discrete and mixed optimization problems.

In structural optimization problems, where the objective function and the constraints are particularly highly non-linear functions of the design variables, the computational effort spent in gradient calculations is usually large.

- Particle Swarm Optimization Algorithm (PSO Method): It was proposed by Kennedy and Eberhart (1995) and it is based on the behavior reflected in flocks of birds, bees and fish that adjust their physical movements to avoid predators and seek for food. PSO has been found to be highly competitive for solving a wide variety of optimization problems. It can handle nonlinear, non-convex design spaces with discontinuities.
- Harmony Search (HS Algorithm): It was originally inspired by the improvisation process of Jazz musicians. According to the analogy between improvisation and optimization, each musician corresponds to each decision variable, while musical instrument's pitch range corresponds to decision variable's value range, musical harmony at certain time corresponds to solution vector at certain iteration and audience's aesthetics corresponds to objective function. Just like musical harmony is improved time after time, the solution vector is improved iteration by iteration.
- Differential Evolution (DE Method): In this work, DE Method is used and it is described in detail below.

4.6 DIFFERENTIAL EVOLUTION

Differential Evolution (DE) is a global optimization technique, developed by Storm and Price (1995). DE belongs to a special kind of differential operator, in order to create new

offspring from parent chromosomes instead of classical crossover or mutation. Even with its short history, DE has been applied to complex problems like Robot design, Aerodynamic design, Concrete beam reinforcement design, optimum design of shell and tube type heat exchanger and Parallel Machine Scheduling.

DE utilizes a population of *NP* parameter vectors $\mathbf{s}_{i,g}$ (i=1,2,...,*NP*) for each generation *g*. It generates new vectors by adding the weighted difference vector between two population members to a third member. If the resulting vector corresponds to a better objective function value than a population member, the newly generated vector replaces this member. The comparison is performed between the newly generated vector and all the members of the population, except the three ones used for its generation. The best parameter vector $\mathbf{s}_{best,g}$ is evaluated in every generation, so as to keep track of the progress achieved during the optimization process.



Figure 4.5 Flowchart of the differential evolution algorithm

If there is no information about the system, the initial population is chosen randomly. As a rule, a uniform distribution for all random decisions will be assumed, unless otherwise stated. If a preliminary solution is available, the initial population is usually generated by adding normally distributed random deviations to the nominal solution $s_{nom,0}$.

Several variants of DE have been tested, the two most promising of which are presented detailed.

4.6.1 Scheme DE1

In the first variant, a donor vector $\mathbf{v}_{i,g+1}$ is generated first according to:

$$\mathbf{v}_{i,g+1} = \mathbf{s}_{r_{1},g} + F \cdot (\mathbf{s}_{r_{2},g} - \mathbf{s}_{r_{3},g}) \tag{4.6}$$

before the computation of the i^{th} parameter vector $\mathbf{s}_{i,g+1}$, where:

- integers r_1 , r_2 and r_3 are chosen randomly from the interval [1, NP] while $i \neq r_1$, r_2 and r_3 .
- *F* is a real constant value, called *mutation factor*, which controls the amplification of the differential variation (**s**_{r2,g}-**s**_{r3,g}) and is defined in the range [0,2].

In the next step the crossover operator is applied by generating the trial vector $\boldsymbol{u}_{i,g+1} = [u_{1,l,g+1}, u_{2,l,g+1}, \dots, u_{D,l,g+1}]^T$ which is defined from the elements of the vector $\boldsymbol{s}_{i,g}$ and the elements of the donor vector $\boldsymbol{v}_{i,g+1}$ whose elements enter the trial vector with probability *CR* as follows:

$$u_{j,i,g+1} = \begin{cases} v_{j,i,g+1} \text{ if } \operatorname{rand}_{j,i} \leq CR \text{ or } j = I_{rand} \\ s_{j,i,g} \text{ if } \operatorname{rand}_{j,i} > CR \text{ or } j \neq I_{rand} \end{cases}$$
(4.7)

where:

- rand_{j,i}~U[0,1],
- I_{rand} is a random integer from [1,2,...,n] that ensures that **v**_{i,g+1}≠**s**_{i,g}.

For example, a certain sequence of the vector elements of u is identical to the elements of **v**, the other elements of u acquire the original values of $\mathbf{s}_{i,g}$. Choosing a subgroup of parameters for mutation, is similar to a process known as *crossover* in evolution theory. This is shown in Figure 4.5 for n=7, where CR \in [0,1] is the crossover probability and constitutes a control variable for the Scheme DE1.

The last step of the generation procedure is the implementation of the selection operator, where the vector $\mathbf{s}_{i,g}$ is compared to the trial vector $\mathbf{u}_{i,g+1}$:

$$\mathbf{s}_{i,g+1} = \begin{cases} \mathbf{u}_{i,g+1} \text{ if } f(\mathbf{u}_{i,g+1}) \le f(\mathbf{s}_{i,g}) \\ \mathbf{s}_{i,g} \text{ otherwise} \end{cases}$$
(4.8)

where i=1,2,...,NP.



Parameter vector containing the parameters \mathbf{s}_{i} , j=1,2,...,n

Figure 4.6 Illustration of the crossover process for n=7

4.6.2 Scheme DE2

In the second variant the donor vector $\mathbf{v}_{i,g+1}$ is generated first according to the relation:

$$\mathbf{v}_{i,g+1} = \mathbf{s}_{i,g} + \lambda \cdot (\mathbf{s}_{best,g} - \mathbf{s}_{i,g}) + F \cdot (\mathbf{s}_{r_{2},g} - \mathbf{s}_{r_{3},g})$$
(4.9)

before the computation of the ith parameter vector $s_{i,g+1}$, by introducing an additional control variable λ . The idea behind λ is to provide a means to enhance the greediness of the scheme by incorporating the current best vector $s_{\text{best},g}$. The generation of the trial vector $u_{i,g+1}$, the construction of u from v and $s_{i,g}$, as well as the decision process are identical to those of DE1.

The DE method for minimizing continuous space functions is shown to be superior to Adaptive Simulated Annealing [ASA] proposed by Ingber (1993) as well as the Annealed Nelder & Mead approach [ANM].^[7] DE was the only technique to converge for all of the functions in the test function suite. For those problems where ASA or ANM could find the minimum, DE usually converged faster, especially in the more difficult cases. Since DE is inherently parallel, a further significant speedup can be obtained if the algorithm is executed on a parallel machine or a network of computers. This is significant especially for real world problems where computing the objective function requires much time. Despite these already promising results, DE is still in its infancy and can most probably be improved. Whether or not an annealed version of DE, or the combination of DE with other optimization approaches is of practical use, is still unanswered. Finally, it is important for practical applications to gain more knowledge on how to choose the control variables for DE.

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LIFE-CYCLE COST ANALYSIS

5.1 INTRODUCTION

Life-cycle cost analysis (LCCP) is a method of determining the entire cost of a product, an enterprise or a structure over its expected useful life. The life-cycle cost of a building consists of the sum of the cost of its present value, plus all the expenses for operating and maintaining the building during its expected lifetime. LCCA is an important procedure in the scientific field of structural engineering, as:

- It can be used as an assessment tool of the response of the building during its expected lifetime,
- It evaluates its influence on the structure in economic terms,
- It constitutes an evaluation tool for making responsible decisions about maintaining, improving and constructing facilities.

In general, the life-cycle cost is related to the possible losses due to deficient performance of the structure under loadings with random occurrence and intensity during its life. The design process should take into account both direct economic and human life losses within a given social context. In order to take into account damage and other earthquake losses into the LCCA procedure, a reliable tool for estimating the capacity of any structural system in multiple earthquake hazard levels is required.

IDA is proven to be an analysis procedure for obtaining satisfactory estimates of the structural performance in the case of earthquake hazards. It is considered, among others, an appropriate method to be incorporated into the LCCA procedure. In view of the complexity and the computational effort required by the 3D structural analysis models simplified 2D structural simulations are usually used during the design procedure. This is mainly justified in plan-symmetric buildings and mostly in the case of steel framed buildings composed by 2D moment resisting frames. In 3D RC buildings, however, the columns belong to two or more intersecting lateral-force-resisting systems, therefore it is not possible to implement a 2D simulation since the bidirectional orthogonal shaking effects are significant and should be taken into account. Moreover, 3D models should also be considered in the case of non-symmetric in plan steel or RC buildings.

5.2 LITERATURE SURVEY

The principles of Life-Cycle Cost Analysis are based on economic theories and have been used as decision-support tools in industrial and commercial projects. LCCA is mainly applied to energy and water conservation projects, as well as transportation projects, including highways, bridges and pavements. For the case of buildings, the application of LCCA is very important, particularly for retrofitted/deteriorating structures. Especially when it comes to seismic regions, LCCA is applied as a structural performance criterion, for taking into account future damages due to earthquakes, as well as a decision-making tool for the most cost-effective solution related to the construction of a building.

The introduction of LCCA was made in the early 1960s, in the field of infrastructures, as an absolute investment assessment tool. In the 1990s in USA, in view of large losses due to earthquakes and hurricanes, there was a need for new design procedures of facilities that protect life and reduce damage and economical impact to an acceptable level.

Beck et al. (2003) introduced a measure, to be incorporated into the seismic risk assessment framework for economic decision-making of buildings, denoted as the probable frequent loss, which is defined as the mean loss resulting from shaking with 10% exceedance probability in 5 years. Liu et al. (2003) suggested a two-objective optimization procedure for designing steel moment resisting frame buildings within a performance-based seismic design framework, where initial material and lifetime seismic damage costs are treated as two separate objectives. Lagaros et al. (2006) have adopted the limit state cost, in order to compare descriptive and performance based design procedures. Frangopol and Liu (2007) reviewed the recent development of life-cycle maintenance and management planning for deteriorating civil infrastructure, with emphasis on bridges. Kappos and Dimitrakopoulos (2008) implemented decision making tools, namely cost-benefit and life-cycle cost analyses, in order to examine the feasibility of strengthening reinforced concrete buildings. Pei and Van De Lindt (2009) proposed a probabilistic framework in order to estimate long-term earthquake-induced economical loss for wood frame structures.

5.3 LIFE-CYCLE COST ANALYSIS OF STRUCTURES

The total cost C_{TOT} of a structure, may refer either to the design-life period of a new structure or to the remaining life period of an existing or retrofitted structure. This cost can be expressed as a function of time and the design vector **s**:

$$C_{TOT}(t,s) = C_{IN}(s) + C_{LC}(t,s)$$
 (5.1)

where C_{IN} is the initial cost of a new or retrofitted structure, C_{LC} is the present value of the life-cycle cost, s is the design vector corresponding to the design loads, resistance and material properties that influence the performance of the structural system, while t is the time period. The term *initial cost* of a new structure refers to the cost required for construction. The initial cost is related to the material and the labour cost for the construction of the building which includes concrete, steel reinforcement, labour cost for placement as well as the non-structural component cost, in the case of a RC building. The term "life-cycle cost" refers to the potential damage cost from earthquakes that may occur during the life of the structure. It should be mentioned that in the calculation of CLC a regularization factor is used that transforms the costs in present values. The estimation of the cost of exceedance of the collapse prevention damage state, will vary considerably according to which approach is adopted by the three steps of the life-cycle cost analysis is presented in the flowchart of Figure 5.1.



Figure 5.1 Flowchart of the life-cycle cost analysis framework

5.3.1 Calculation of the life-cycle cost

The calculation of C_{LC} requires at first the detection and quantification of the damage that a structure sustains during an earthquake. Damage may be quantified by using several DIs, whose values can be related to particular structural damage states, also called limit states. The idea of describing the state of damage of the structure by a specific quantity, on a defined scale in the form of a damage index, is simple. Damage, in the context of life-cycle cost assessment, refers not only to structural damage but also to non-structural damage. The latter including the case of architectural, mechanical, electrical and plumbing damage and also the damage of furniture, equipment and other contents. The maximum inter-story drift (θ) has been considered as the response parameter which best characterizes the structural damage, associated with all types of losses. It is generally accepted that inter-storey drift can be used as a reliable limit state criterion to determine the expected damage. On the other hand, the intensity measure which has been associated with the loss of contents, like furniture and equipment, is the maximum response floor acceleration (acc).

Limit State	Inter-storey Drift (%)	Floor Acceleration (g)	
	(Ghobarah, 2004)	(Elenas & Meskouris, 2001)	
(I) - None	θ≤0.1	Ófloor≤0.05	
(II) - Slight	0.1<θ≤0.2	<0.05ófloor≤0.10	
(III) - Light	0.2<θ≤0.4	0.10<ófloor≤0.20	
(IV) - Moderate	0.4<θ≤1.0	0.20<ófloor≤0.80	
(V) - Heavy	1.0<θ≤1.8	0.80<ófloor≤0.98	
(VI) - Major	1.8<θ≤3.0	0.98<ófloor≤1.25	
(VII) - Collapsed	θ>3	Ófloor>1.25	

Table 5.1 Damage indices limits for bare moment resisting frames

Considering the way that the potential damage is detected, the life-cycle cost (C_{LC}) configuration involves the sum of functions time of the corresponding costs based on different *DI*, where *DI* is the number of the damage indices that used to quantify the damage of a structure (Eq. 4.2b). Each damage cost based on a *DI* is a function of the sum of the limit state costs (C_{LS}), since each *DI* is quantified in six limit states, where n is the number of the considered limit states, which differ according to the damage index:

$$C_{LC}(t,s) = \sum_{i=1}^{DI} C_{LS}^{i}(t,s)$$
(5.2a)
$$C_{LS}^{DI}(t,s) = \sum_{i=1}^{n} f(C_{LS}^{i},t,s)$$
(5.2b)

The C_{LS} accounts for the cost of repair, the cost of loss of contents related to loss of contents, rental and income, after an earthquake. The quantification of these losses in economical terms depends on several socio-economic parameters. The most difficult cost to quantify is the cost corresponding to the loss of a human life. There are a number of approaches for its estimation, ranging from purely economic reasoning to more sensitive that consider the loss of a human being irreplaceable.

The limit state cost (C_{LS}) , for the i-th limit state, can thus be expressed as follows:

$$C_{LS}^{i} = C_{dam}^{i} + C_{con}^{i} + C_{inc}^{i} + C_{inj}^{i} + C_{fat}^{i}$$
(5.3a)

$$C_{con}^{i} = C_{con}^{i,\theta} + C_{con}^{i,acc}$$
(5.3b)

where C_{dam}^{i} is the damage repair cost, $C_{con}^{i,\theta}$ is the loss of contents cost due to structural damage that is quantified by the maximum inter-storey drift, while $C_{con}^{i,acc}$ is the loss of contents cost due to floor acceleration, C_{ren}^{i} is the loss of rental cost, C_{inc}^{i} is the income

loss cost, C_{inj}^i is the cost of injuries and C_{fat}^i is the cost of human fatality. These cost components are related to the damage of the structural system.

Cost Category	Calculation Formula	Basic Cost		
Damage (repair (C+++)	Replacement cost × floor area × mean			
Damage/Tepair (Cdam)	damage index	1500 MU/m ²		
Loss of contonts (Com)	Unit contents cost × floor area × mean			
LOSS OF CONTENTS (Ccon)	damage index	500 MU/m ²		
Pontal (Cruz)	Rental rate × gross leasable area × loss of			
Refital (Cren)	function	10 MU/month/m ²		
Incomo (Cirro)	Rental rate × gross leasable area × down			
	time	2000 MU/year/m ²		
	Minor injury cost per person × floor area ×			
Minor Injury (Cinj,m)	occupancy rate × expected serious injury			
	rate	2000 MU/person		
Serious Injury (Circia)	Serious injury cost per person × floor area ×			
Serious injury (Cinj,s)	occupancy rate × expected death rate	2 × 10 ⁴ MU/person		
Human Fatality (C _{fat})	Human fatality cost per person × floor area ×	2.8 × 10 ⁶		
	occupancy rate × expected death rate	MU/person		
* Occupancy rate 2 persons/100 m ²				

Table 5.2 Limit state cost – calculation formulas

Table 5.3 provides the ATC-13 (1985) and FEMA-227 (1992) limit state dependent damage consequence severities.

	FEMA-227 (1992)				ATC-13 (198	5)
Limit State	Mean damage index (%)	Expected minor injury rate	Expected serious injuries rate	Expected death rate	Loss of function (%)	Down time (%)
(I) - None	0	0	0	0	0	0
(II) - Slight	0.5	3.0E-05	4.0E-06	1.0E-06	0.9	0.9
(III) - Light	5	3.0E-04	4.0E-05	1.0E-05	3.33	3.33
(IV) - Moderate	20	3.0E-03	4.0E-04	1.0E-04	12.4	12.4
(V) - Heavy	45	3.0E-02	4.0E-03	1.0E-03	34.8	34.8
(VI) - Major	80	3.0E-01	4.0E-02	1.0E-02	65.4	65.4
(VII) - Collapsed	100	4.0E-01	4.0E-01	2.0E-01	100	100

Table 5.3 Limit state parameters for cost evaluation

Based on a Poisson process model of earthquake occurrences and an assumption that damaged buildings are immediately retrofitted to their original intact conditions after each major damage-inducing seismic attack, Wen and Kang (2001) proposed the following formulae for the limit state cost function considering *N* limit states:

$$C_{LS}^{DI}(t,s) = C_{LS}^{\theta}(t,s) + C_{LS}^{acc}(t,s)$$
(5.4a)

$$C_{LS}^{\theta}(t,s) = \frac{\nu}{\lambda} \left(1 - e^{-\lambda t}\right) \sum_{i=1}^{N} C_{LS}^{i,\theta} \cdot P_i^{\theta}$$
(5.4b)

$$C_{LS}^{acc}(t,s) = \frac{\nu}{\lambda} \left(1 - e^{-\lambda t}\right) \sum_{\iota=1}^{N} C_{LS}^{i,acc} \cdot P_i^{acc}$$
(5.4c)

where

$$P_i^{DI} = P(DI > DI_i) - P(DI > DI_{i+1})$$
(5.5)

and

$$P(DI > DI_i) = \left(-\frac{1}{t}\right) \cdot \ln[1 - \overline{P}_i(DI - DI_i)]$$
(5.6)

 P_i is the probability of the ith limit state being violated given the earthquake occurrence, C_{LS}^i is the corresponding limit state cost, $P(DI - DI_i)$ is the exceedance probability given occurrence, DI_i, DI_{i+1} are the damage indices (maximum inter-storey drift or maximum floor acceleration) defining the lower and upper bounds of the ith limit state, $\overline{P}_i(DI - DI_i)$ is the annual exceedance probability of the maximum damage index DI_i , v is the annual occurrence rate of significant earthquakes modeled by a Poisson process and t is the service life of a new structure or the remaining life of a retrofitted structure.^[4] Thus, for the calculation of the limit state cost of Eq. (5.4b) the maximum floor acceleration is used. The first component of Eqs. (5.4b) or (5.4c), with the exponential term, is used in order to express C_{LS} in present value, where λ is the annual monetary discount rate. In this dissertation the annual monetary discount rate is accurate enough for all practical purposes.

Each limit state is defined by the drift ratio limits or the floor acceleration, as listed in Table 5.1. When one of the *DIs* is exceeded, the corresponding limit state is assumed to be reached. The annual exceedance probability $\overline{P}_i(DI > DI_i)$ is obtained from a relationship of the form:

$$\overline{P}_{l}(DI > DI_{i}) = \gamma(DI_{i})^{-k}$$
(5.7)

where the parameters γ and k are obtained by best fit of known $\overline{P_i} - DI_i$ pairs for each of the two *DIs*. According to Poisson's, the annual probability of exceedance of an earthquake with a probability of exceedance *p* in *t* years is given by the relationship:

$$\bar{P} = \left(-\frac{1}{t}\right) \cdot \ln(i-p) \tag{5.8}$$

This means that the 2/50 earthquake has a probability of exceedance equal to $\overline{P_{2\%}} = -\frac{\ln(1-0.02)}{50} = 4.04 \times 10^{-4} (4.04 \times 10^{-2}\%).$

5.3.2 Implementation of the analysis procedures in the LCCA framework

The limit state cost calculation procedure requires the assessment of the structural capacity in at least three hazard levels of increased intensity with the definition of at least three pairs of annual probability of exceedance (\overline{P}_l) and maximum value of the damage index in question (DI_i) . In this work the abscissa values of the $(\overline{P}_I - DI_i)$ pairs, corresponding to the maximum values of the damage index, are obtained either by means of nonlinear static or dynamic analysis procedure, while the ordinate values correspond to the annual probabilities of exceedance. These probabilities correspond to discrete values of annual probabilities of exceedance obtained from a hazard curve, which describe the seismic risk of a region.

The implementation of results of a structural analysis in the LCCA procedure depends on the type of the analysis implemented (incremental nonlinear static or incremental nonlinear dynamic). Significant role in that step of the LCCA framework plays the selection procedure of the seismic actions.



Figure 5.2 Hazard curve of the city of San Diego, California (Latitude (N) 32.7°, Longitude (W) 117.2°)

 Nonlinear static analysis procedure: For the implementation of the NDA, multiple Nonlinear dynamic analyses have to be performed in order to assess the structural performance in the selected hazard levels. For each hazard level a number of seismic records is selected and the median response among the records is calculated. Therefore, the application of NDA incorporated into LCCA results in a timeconsuming and computationally-demanding procedure compared to the corresponding NSA implementation. From this procedure a scale factor is calculated for each one of the ground motions and for each hazard level. In order to preserve the relative scale of the two components of the records in the longitudinal and transverse directions, the component of the record having the highest intensity measure is scaled first, while a scaling factor that preserves their relative ratio is assigned to the second component (Figure 5.3). More details on the nonlinear dynamic analysis procedures (IDA, MIDA and MSDA) are provided in Chapter 3 of the thesis.



Figure 5.3 Implementation of nonlinear static analysis procedure in LCCA framework

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NUMERICAL TESTS

6.1 INTRODUCTION

For the purposes of this study two test examples are considered. In particular a three storey and a six storey 3D symmetrical RC buildings are used. For each building performance-based optimization problems are formulated and the optimum designs obtained through the different formulations are critically assessed. More specifically both test examples are optimally designed considering fixed support conditions as well as with base isolation. In the latter case Lead Rubber Bearings (LRB) and High-Damping Natural Rubber Bearings (HDNR) isolation systems are examined. For the critical assessment the life-cycle cost analysis procedure is implemented.

6.2 DESCRIPTION OF THE STRUCTURAL MODELS

The three storey RC building, shown in Figure 6.1, and the six storey RC building, shown in Figure 6.2, have been considered in order to perform the critical assessment of the design procedure, where the height of the first floor is 4m, while the height of the other floors is 3m. Concrete of class C20/25 (nominal cylindrical strength of 20MPa) and class S500 steel (nominal yield stress of 500MPa) are assumed. The slab thickness is equal to 18 cm and is considered to contribute to the moment of inertia of the beams with an effective flange width. In addition to the self-weight of the beams and the slab, a distributed dead load of 2 kN/m², due to floor finishing and partitions and imposed live load with nominal value of 1.5 kN/m², is considered, in the combination with gravity loads ("persistent design situation"). Nominal dead and live loads are multiplied by load factors of 1.35 and 1.5, respectively. Following EC8, in the seismic design combination, dead loads are considered with their nominal value, while live loads with 30% of their nominal value. Moreover, the strong-column weak-beam guideline was followed in the design process.



Figure 6.1 Front view of the three storey non-symmetrical test example



Figure 6.2 Front view of the six storey non-symmetrical test example

6.3 MODELLING AND FINITE ELEMENT ANALYSIS

The simulation of the beam – column joints using rigid elements was accomplished by specifying peripheral nodes to the column axis. The definition of the nodes is clear, expressing the logic sequence of them. Afterwards, the main nodes were connected to the peripheral nodes with elements, which are chosen to have very big area and moments of inertia (very stiff elements) so as to accomplish the rigid link between the column axis and the edge points of the beams, in a way which is unaffected from the dimensions of the column sections.

Each nonstructural elements of the model was simulated with beam elements (each node 12 degrees of freedom). A fiber approach (Figure 6.3) was used for the section simulation of columns and an elastic section for beams. For the nonlinear and linear procedures nonlinear element and linear elements where used for the columns' simulation respectively. For the nonlinear elements three and four points of integration where used for the simulation of beams and columns, respectively.



Figure 6.3 Modeling of inelastic behavior – the fiber approach

In order to perform the nonlinear static or dynamic analysis of frame structures two type of models are used: lumped and distributed plasticity models. In the distributed plasticity models the plasticity is evaluated in a number of sections along the element which in turn are divided to a number of monitoring points which correspond to the fibers of the section. The distributed plasticity model is also known as the fiberapproach. The use of fibers allows the accurate representation of the stress level across the sections, while using a number of sections along the element allows nonlinearities to develop along the element. The yield criteria adopted for the fiber approach is uniaxial stress-strain laws for each fiber.



Figure 6.4 Behavior of the beams' and columns' materials

Referring to the base nodes they are completely fixed, while the two slabs act as diaphragms and have width 18 cm. The materials that were used are reinforced concrete C20/25 and reinforcing steel S500. Their behavior is depicted in Figure 6.4(a) and Figure 6.4(b), respectively.

The simulation of the bearings was accomplished using a zero length element with a uniaxial material with combine linear kinematic and isotropic hardening depicted in Figure 6.5.



Figure 6.5 Hardening material

6.4 OPTIMUM DESIGN FRAMEWORK OF THE RC FRAMES

The formulation of the optimization problem for the three storey RC frame and the six storey RC frame differs in case of a fixed based as well as an isolated frame. In all six cases of optimum design the initial construction cost is the objective function to be minimized. The design variables are the dimensions of columns and beams cross-sections and the longitudinal reinforcement of the beams and the columns for the fixed based RC frames plus the mechanical characteristics of the isolation system for the isolated RC frames.

6.4.1 Formulation of the optimization problem of the fixed based RC frames

Objective function

As mentioned above, the objective function to be minimized for both fixed frames is the initial construction cost:

$$minC_{IN} = C_{BEAMS} + C_{COLUMNS} \tag{6.2}$$

Constraint functions

The *constraint functions* are the maximum interstorey drifts (θ_{max}) of the superstructure:

PBD f	fixed
50% / 50y	$\theta_{\rm max} < 0.4\%$
10% / 50y	$\theta_{\rm max}$ <1.8%
2%/50y	$\theta_{\rm max} < 3.0\%$

Design Variables

The two dimensions of the columns/beams along with the longitudinal reinforcement constitute the design variables. In the three storey frame there are five groups of design variables, three groups for columns and two for beams for all three floors. These are shown in Table 6.1 and Figure 6.6. For the case of the six storey RC frame the design variables are separated in ten groups, three for the columns and two for the beams or the three first floors and three for the columns, two for the beams for the three final floors as presented in Table 6.2 and Figure 6.7.

Groups	Structural Elements	
1 st	C1,C4, C9, C12	ast class
2 nd	C2, C3, C5, C8, C10, C11	
3 rd	C6, C7	2 rd floor
4 th	B1, B2, B3, B8, B9, B10, B15, B16, B17	5 11001
5 th	B4, B5, B6, B7, B11, B12, B13, B14	-

Table 6.1 Design variables groups for the 3 strorey RC frame

Table 6.2 Design variables groups for the 6 strorey RC frame

Groups	Structural Elements	
1 st	C1,C4, C9, C12	1 st floor
2 nd	C2, C3, C5, C8, C10, C11	to
3 rd	C6, C7	3 rd floor
4 th	C37, C40, C45, C48	4 th floor
5 th	C38, C39, C41, C44, C46, C47	to
6 th	C42, C43	6 th floor
7 th	B1, B2, B3, B8, B9, B10, B15, B16, B17	1 st floor to
8 th	B4, B5, B6, B7, B11, B12, B13, B14	6 th floor



Figure 6.6 Plan of the first level of the buildings



Figure 6.7 Plan of the fourth level of the six-storey

6.4.2 Formulation of the optimization problem of the isolated RC frames

Objective function

The *objective function* to be minimized for both isolated buildings is the initial construction cost:

$$minC_{IN} = C_{IN,BEAMS} + C_{IN,COLUMNS} + C_{IN,ISOLATORS}$$
(6.3)

Constraint functions

The *constraint functions* of the isolated frames arise from the preliminary and the final design, as described in Chapter 2:

- Minimum lateral displacement $D_D < 500mm$,
- Maximum interstorey drift limit

$$\theta_{max} \le \frac{0.01}{R_1} \le \begin{cases} 2.5\% \left(For \, T_{fixed} < 0.7 \text{sec} \right) \\ 2\% \left(For \, T_{fixed} > 0.7 \text{sec} \right) \end{cases}$$
(6.4)

- Base shear of the superstructure check $V_s \approx V_{codeshear} = m_{tot} \cdot SA(T_1)$ (6.5) where V_s is the minimum lateral design force of the superstructure, calculated in Chapter 2, $V_{codeshear}$ is the base shear resulted from the elastic analysis and $SA(T_1)$ results from the response spectrum of the fixed superstructure.
- Maximum interstorey drift limits for two for the DBE and MCE of the *PBD isolated*

- Buckling check of each isolator $SF \ge SF_{target}$
- Lateral displacement check of each isolator:

circular
$$D_{crit} = 2R \cdot \frac{\pi}{4} \left[1 - \left(\frac{P}{P_{crit}}\right)^2 \right], \quad P_{crit} = \frac{\pi}{\sqrt{6}} \cdot S \cdot S_2 \cdot G$$
 (6.6)

Design Variables

In addition to the design variables considered for the fixed building mentioned above, i.e. the dimensions of the columns and beams cross sections (Figure 6.6, 6.7) and the longitudinal reinforcement, the dimensionless characteristic strength a, which affects the mechanical characteristics of the bearings is considered.

6.4.3 Structural analysis

The analysis applied for the purpose of the optimum design of the two fixed based RC frames as well as the four isolated RC frames obtained is the Nonlinear Static Procedure (NSP), which is described in Chapter 3, performed using the program OpenSees. The

structural model is "pushed" according to a predefined lateral load pattern. The whole procedure is based on the assumption that the response is related to the response of an equivalent single degree of freedom system. The lateral loads are imposed in conjunction with the imposition of gravity loads.

The structure is "pushed" under lateral loads, which are increased proportionally, until the target displacement is reached from a characteristic node of the structure model, or earlier if the algorithm fails to converge because a collapse mechanism has been formed. In the case of the fixed based RC frames the target displacement for each hazard level is calculated using the target displacement method (Chapter 3, § 3.3.2) and the response spectrums for the three corresponding hazard levels; with 2%, 10% and 50% probability of exceedance (Figure 6.8).

The base shear is obtained from the EC8 elastic response spectrum for soil type B (characteristic periods TB=0.15 sec and TD=2.00 sec), while the importance factor γ I was taken equal to 1.0, as well as the behavior factor of the structure q. PGA is taken from Table 6.3 for three hazard levels with 2%, 10% and 50% probability of exceedance in 50 years.

Event	Recurrence Interval	Probability of Exceedence	PGA (g)
Occasional	72 years	50% in 50 years	0.11
Rare	475 years	10% in 50 years	0.31
Very Rare	2475 years	2% in 50 years	0.78

 Table 6.3 PGA according to the frequency of the seismic hazard



Figure 6.8 Elastic design spectra with 2%, 10% and 50% probabilities of exceedance for the fixed buildings

For the isolated building with LRB and HDNR the nonlinear static analysis under was performed based on the elastic response spectrums in Figure 6.6 and the target displacements where derived from the equations 2.18. in Chapter 2.



The elastic design spectra of the isolated buildings are shown in Fig. 6.9 and 6.10.

Figure 6.9 Elastic design spectra with probability of exceedance 10% for damping factors β =5% β =10% and β =20%

In Figures 6.9 and 6.10 are presented the response spectrum for the fixed, isolated HDNR and isolated LRB for the corresponding damping factors β =5%, β =10%, β =20% respectively. In this work along the optimum design process the structural elements of the superstructure and its fundamental period are changing until the optimum achieved. In Figures 6.9 and 6.10 are depicted the elastic design spectrum in case of a fixed frame, an isolated with HDNR and an isolated with LRB for the corresponding damping factors. Based on the assumption that along the optimum design procedure the fundamental period of the structure is T1_fixed=0.6 sec the dotted lines are referring to the allowable regions of design acceleration for the isolated systems. For example in the case of 10% probability of exceedance and for T1_fixed=0.6 sec, the fundamental period for the isolated structure is 3 times T1_fixed, which is equal to T1_iso=1.8 sec. In that case the allowable range of acceleration values is lower than 2.4 m/sec2 and 3.0 m/sec2, for damping factors β =20% and β =10%, respectively.



Figure 6.10 Elastic design spectra with probability of exceedance 2% for damping factors β =5% β =10% and β =20%

6.4.4 Optimum designs

The dimensions of the components (i.e. columns, beams, reinforcement, bearings) arose from the NSP and are shown in Tables 6.4 to 6.9, while the plans of the structures are illustrated in Figures 6.6 and 6.7.

The intersections 1 in the three-storey buildings refer to the four corner columns, the intersections 2 to the rest external ones and the intersections 3 to the internal columns. The same numbering refers to the lower three levels of the six-storey, while in the upper three levels the numbering of the intersections is 4, 5 and 6, respectively. The intersections x refer to the horizontal beams and the intersections y to the vertical ones, for both examples.

	Fixed Building		
	h i (m)	b i (m)	reinfi
1	0,50	0,60	0,010
2	0,60	0,40	0,010
3	0,45	0,40	0,0116
x	0,35	0,30	0,010
у	0,55	0,35	0,010

Table 6.4 Dimensions of the structural elements of the three storey fixed RC fame
_	Isolated Building			
	h i (m)	reinfi		
1	0,55	0,70	0,011007	
2	0,60	0,40	0,010993	
3	0,45	0,40	0,013404	
х	0,65	0,35	0,010483	
у	0,65	0,35	0,010847	
D=0,35	d=0,15	h=0,25	α=8,2	

Table 6.5 Dimensions of the structural elements of the three storey isolated RC fame with LRB

Table 6.6 Dimensions of the structural elements of the three storey isolated RC fame with HDNR

	Isolated Building			
	h i (m)	b i (m)	reinfi	
1	0,50	0,55	0,012657	
2	0,50	0.30	0,010839	
3	0,40	0.35	0,012219	
х	0,30	0,25	0,01061	
у	0,55	0,30	0,010216	
D=	0,4 h	=0,25	α=3,8	

Table 6.7 Dimensions of the structural elements of the six storey fixed RC frame

		Fixed Building			
	h _i (m)	b _i (m)	reinfi		
1	0,55	0,70	0,011007		
2	0,60	0,40	0,010993		
3	0,45	0,40	0,013404		
4	0,40	0,35	0,011660		
5	0,40	0,35	0,011866		
6	0,40	0,35	0,011088		
х	0,65	0,35	0,010483		
у	0,65	0,35	0,001085		

Table 6.8 Dimensions of the structural elements of the six storey isolated RC fame with LRB

	lso	Isolated Building			
	hi (m)	bi (m)	reinfi		
1	0,45	0,60	0,011386		
2	0,60	0,40	0,010498		
3	0,45	0,35	0,015549		
4	0,30	0,30	0,013320		
5	0,40	0,35	0,014537		
6	0,40	0,35	0,012117		
x	0,55	0,35	0,010593		
У	0,50	0,35	0,011154		
D=0,5	d=0,2	h=0,25	α=6,5		

	Isolated Building			
	h i (m)	b i (m)	reinfi	
1	0,45	0,80	0,011988	
2	0,55	0,40	0,011028	
3	0,40	0,40	0,015771	
4	0,35	0,35	0,012013	
5	0,40	0,35	0,012348	
6	0,35	0,35	0,014285	
х	0,35	0,35	0,014835	
У	0,60	0,35	0,013649	
D=0,5	h=	0,25	α=3,4	

Table 6.9 Dimensions of the structural elements of the six storey isolated RC fame with HDNR

In Table 6.10 the initial cost of the final optimum designs is presented, in the case of the fixed and the isolated frames.

Docign	Superstructure		Foundation		Total cost		
Design	Steel	Concrete	Infills	Foundation	Isolation		
Frame 3 fixed	62612	58272	22365	35033	0	178282	
Frame 6 fixed	128379	129153	22155	70918	0	350605	
Frame 3 iso_LRB	62077	62097	22750	58985	27000	232910	
Frame 3 iso_HDNR	124881	124608	22470	118473	54000	444431	
Frame 6 iso_LRB	63874	64494	22925	61052	29100	241445	
Frame 6 iso_HDNR	128455	124055	22295	119392	45000	439197	

Table 6.10 Initial cost of the optimum designs (in euros)

6.5 LIFE CYCLE COST ASSESSMENT OF THE OPTIMUM DESIGNS

The final optimum designs are assessed conducting life cycle cost analysis. For the purpose of a life cycle cost analysis (Chapter 5), values of the maximum interstorey drift and the maximum floor acceleration should be calculated for three hazard levels (2%/50y, 10% /50y and 50%/50y) for each optimum design obtained.

The performance of each optimum design for each hazard level derived performing a Multi-stripe Incremental Dynamic Analysis (MSDA) using three natural records for each hazard level shown in Table 6.11, 6.12, 6.13. The procedure is described in paragraph 5.3.2 where a scale factor is calculated for each one of the natural records and for each hazard level. In order to preserve the relative scale of the two components of the records in the longitudinal and transverse directions, the component of the record having the highest intensity measure is scaled first, while a scaling factor that preserves their relative ratio is assigned to the second component.

()	Earthquake	Station	Distance	Site	
Ĕ	Cape Mendocino	Butler Valley	37	rock	
C fra	(CM)	Eureka School	24	soil	
Ř	25 April 1992				
rey	Cape Mendocino				
sto	(C1)				
e	aftershock, 26 April	Ferndale	34	soil	
Ъг	1992				
	0741GMT				
	Earthquake	Station	Distance	Site	
	Cape Mendocino		37	rock	
	(CM)	Butler Valley			
e	25 April 1992	Dutiel Valley			
am					
L L	Cape Mendocino				
, RC	(C1)				
re)	aftershock, 26 April	Ferndale	34	soil	
sto	1992				
Xia	0741GMT				
01	Cape Mendocino				
	(C2)	Forndalo	21	soil	
	aftershock, 4/26/92	remuale	34	3011	
	1118GMT				

Table 6.11 Natural records representing the 50% in 50 year hazard level for the three storey andsix storey RC frame

Table 6.12 Natural records representing the 10% in 50 year hazard level for the three storey andsix storey RC frame

	Earthquake	Station	Distance	Site
Three storey RC frame	Tabas (TB) 16 September 1978	Tabas	1.1	rock
	Cape Mendocino (CM) 25 April 1992	Cape Mendocino	6.9	rock
	Chi-Chi (CC), Taiwan 20 September 1999	TCU078	6.9	soil
	Earthquake	Station	Distance	Site
Six storey RC frame	Cape Mendocino (CM) 25 April 1992	Petrolia	8.1	soil
	Chi-Chi (CC), Taiwan	TCU067	2.4	soil
	20 September 1999	TCU074	12.2	soil

me ne	Earthquake	Station	Distance	Site
H Valparaiso (VL), Chile	Vina del Mar	30	soil	
	Llollea	34	rock	
	Michoacan (MI),			
	Mexico	La Union	22	rock
	19 September 1985			

Table 6.13 Natural records representing the 2% in 50 year hazard level for the three storey andsix storey RC frame

For the calculation of the initial cost values of the material and the labour cost for the construction of the building which includes concrete, steel reinforcement, labour cost for placement as well as the non-structural component cost, in the case of a RC structure were used 2.5 \leq /kg for the steel reinforcement, 100 \leq /m³ for the concrete, 35 \leq /m² for the infills and 60 \leq /lt for the bearings. The cost of the bearings include the purchase, the installation and the cost of the custom office.

6.6 NUMERICAL RESULTS OF THE THREE STOREY OPTIMUM DESIGNS

Comparing the results of the analysis of the isolated models to the fixed model, significant decrease in the maximum interstorey drift and accelerations are observed (Figures 6.11-6.34). As far as the maximum interstorey drift concerned, the reduction observed in the design with LRB (D_{LRB}) is almost the same with the one observed in the design with HDNR (D_{HDRN}), compared to the fixed design (D_{fixed}). It is noticed that the two isolated structures present a more flexible response where the frequency of the acceleration motion is lower compared to this of the fixed structure. Clearly, reduced accelerations provide significant reduction in the seismic design forces and hence reduce the risk of structural and non-structural earthquake damage.



Figure 6.11 Time history of the roof acceleration of the fixed and isolated three storey RC frames (Michoacan, Mexico-La Union, x direction)



Figure 6.12 Time history of the roof acceleration of the fixed and isolated three storey RC frames (Michoacan, Mexico-La Union, y direction)



Figure 6.13 Time history of the roof acceleration of the fixed and isolated three storey RC frames (Valparaiso, Chile-Liollea, x direction)



Figure 6.14 Time history of the roof acceleration of the fixed and isolated three storey RC frames (Valparaiso, Chile-Liollea, y direction)



Figure 6.15 Time history of the roof maximum interstorey drift of the fixed and isolated three storey RC frames (Michoacan, Mexico-La Union, x direction)



Figure 6.16 Time history of the roof maximum interstorey drift of the fixed and isolated three storey RC frames (Michoacan, Mexico-La Union, y direction)



Figure 6.17 Time history of the roof maximum interstorey drift of the fixed and isolated three storey RC frames (Valparaiso, Chile-Llollea, x direction)



Figure 6.18 Time history of the roof maximum interstorey drift of the fixed and isolated three storey RC frames (Valparaiso, Chile-Llollea, y direction)



Figure 6.19 Time history of the roof acceleration of the fixed and isolated three storey RC frames (Chi-Chi, Taiwan, x direction)



Figure 6.20 Time history of the roof acceleration of the fixed and isolated three storey RC frames (Chi-Chi, Taiwan, y direction)



Figure 6.21 Time history of the roof acceleration of the fixed and isolated three storey RC frames (Tabas, Tabas, x direction)



Figure 6.22 Time history of the roof acceleration of the fixed and isolated three storey RC frames (Tabas, Tabas, y direction)



Figure 6.23 Time history of the roof maximum interstorey drift of the fixed and isolated three storey RC frames (Chi-Chi, Taiwan, x direction)



Figure 6.24 Time history of the roof maximum interstorey drift of the fixed and isolated three storey RC frames (Chi-Chi, Taiwan, y direction)



Figure 6.25 Time history of the roof maximum interstorey drift of the fixed and isolated three storey RC frames (Tabas, Tabas, x direction)



Figure 6.26 Time history of the roof maximum interstorey drift of the fixed and isolated three storey RC frames (Tabas, Tabas, y direction)



Figure 6.27 Time history of the roof acceleration of the fixed and isolated three storey RC frames (Cape Mendocino, Ferndale-C1, x direction)



Figure 6.28 Time history of the roof acceleration of the fixed and isolated three storey RC frames (Cape Mendocino, Ferndale-C1, y direction)



Figure 6.29 Time history of the roof acceleration of the fixed and isolated three storey RC frames (Cape Mendocino, Ferndale-CM, x direction)



Figure 6.30 Time history of the roof acceleration of the fixed and isolated three storey RC frames (Cape Mendocino, Ferndale-CM, y direction)



Figure 6.31 Time history of the roof maximum interstorey drift of the fixed and isolated three storey RC frames (Cape Mendocino, Ferndale-C1, x direction)



Figure 6.32 Time history of the roof maximum interstorey drift of the fixed and isolated three storey RC frames (Cape Mendocino, Ferndale-C1, y direction)



Figure 6.33 Time history of the roof maximum interstorey drift of the fixed and isolated three storey RC frames (Cape Mendocino, Ferndale-CM, x direction)



Figure 6.34 Time history of the roof maximum interstorey drift of the fixed and isolated three storey RC frames (Cape Mendocino, Ferndale-CM, y direction)

In Figures 6.35-6.43 the maximum drifts between the sequential floors of the threestorey buildings are illustrated. It is observed that the use of seismic isolation highly reduces the drifts of the superstructure, especially in the case of the earthquakes 2%/50 years. Also it is worth mentioning that in the case of the D_{LRB} and D_{HDRN} the interstorey drift presents a uniform distribution trend along the floors in the contrary of the D_{fixed} where the interstorey drift presents a reduced trend from the bottom to the roof. Thus, the superstructure of the D_{LRB} and D_{HDRN} moves along the horizontal directions as a rigid body.



Figure 6.35 Maximum interstorey drifts of the fixed and isolated three storey RC frames (Michoacan, Mexico-La Union)



Figure 6.36 Maximum interstorey drifts of the fixed and isolated three storey RC frames (Valparaiso, Chile-Llollea)



Figure 6.37 Maximum interstorey drifts of the fixed and isolated three storey RC frames (Valparaiso, Chile-Vina del Mar)



Figure 6.38 Maximum interstorey drifts of the fixed and isolated three storey RC frames (Chi-Chi, Taiwan-TCU078)



Figure 6.39 Maximum interstorey drifts of the fixed and isolated three storey RC frames (Cape Mendocino (CM), Cape Mendocino)



Figure 6.40 Maximum interstorey drifts of the fixed and isolated three storey RC frames (Tabas (TB), Tabas)



Figure 6.41 Maximum interstorey drifts of the fixed and isolated three storey RC frames (Cape Mendocino (C1), Ferndale)



Figure 6.42 Maximum interstorey drifts of the fixed and isolated three storey RC frames (Cape Mendocino (CM), Butler Valley)



Figure 6.43 Maximum interstorey drifts of the fixed and isolated three storey RC frames (Cape Mendocino (CM), Eureka School)

Figure 6.44 depicts the optimum designs obtained with reference to the type of foundation (fixed, HDNR, LRB), along with the life cycle cost components calculated for the Ghobarah drift limits. A general observation can be obtained from this figure that design without base isolation is worst compared to the other two designs with respect to the life cycle cost (C_{LC}). Comparing design D_{HDNR} with the design D_{fixed} , it can be seen

that it is 19% cheaper while it is 8% cheaper compared to the design D_{LRB} with reference to the life cycle cost. Also, design D_{LRB} is cheaper 11% compared to the design D_{fixed} with reference to C_{LC} .



Figure 6.44 Three storey test example - Contribution of the initial cost and life cycle cost components to the total cost for different types of foundation





The contribution of the initial and life cycle cost components to the total life-cycle cost are shown in Figure 6.45 .The initial cost (C_{in}) represents the 53% of the total cost for design D_{fixed} while for designs D_{LRB} and D_{HDRN} represents the 63%, 65% respectively.

Although the initial cost is the dominant contributor for all optimum designs, in the case of the life cycle cost components the dominant contributor for all designs also is the loss of contents due to floor acceleration, while damage/repair cost represents the second dominant contributor with reference to the life cycle cost components. In particular the loss of contents components is four times that of the repair cost. Worth mentioning also that the loss of contents contribution due to the maximum interstorey drift is insignificant compared to the contribution due to the floor acceleration. Although the three designs differ significantly injury (minor/major) and fatality costs represent only a small quantity of the total cost: 0.0010% for design D_{fixed} , while for designs D_{LRB} and D_{HDRN} represents the 0.00024%, 0.00026% of the total cost, respectively. Thus, all three designs satisfy the life safety criterion.

Comparing design D_{fixed} with the design D_{HDNR} , it can be seen that D_{fixed} is 26% cheaper while it is 23% cheaper compared to the design D_{LRB} with reference to C_{in} . Also, design D_{LRB} is cheaper 4% compared to the design D_{fixed} with reference to C_{in} .

6.7 NUMERICAL RESULTS OF THE SIX STOREY OPTIMUM DESIGNS

In Figures 6.46-6.51 the maximum values of the interstorey drift component are depicted as well as the maximum values of the roof acceleration component for three seismic records representative for 2%, 10% and 50% probability of exceedance. Comparing the results of the analysis of the isolated structures to the fixed structure, significant decrease in the maximum interstorey drift and accelerations are observed. As far as the maximum roof acceleration concerned, the reduction observed in D_{HDNR} is higher than in D_{LRB} , compared to D_{fixed} . As in the case of the three storey structure, it is noticed that the two isolated structures present a more flexible response where the frequency of the acceleration motion is lower compared to this of the fixed structure.



Figure 6.46 Time history of the roof maximum floor acceleration of the fixed and isolated six storey RC frames (Valparaiso, Chile-Vina del Mar)



Figure 6.47 Time history of the maximum roof interstorey drift of the fixed and isolated six storey RC frames (Valparaiso, Chile-Vina del Mar)



Figure 6.48 Time history of the roof maximum floor acceleration of the fixed and isolated six storey RC frames (Cape Mendocino, Petrolia)



Figure 6.49 Time history of the maximum roof interstorey drift of the fixed and isolated six storey RC frames (Cape Mendocino, Petrolia)



Figure 6.50 Time history of the roof maximum floor acceleration of the fixed and isolated six storey RC frames (Cape Mendocino, Butler Valley)



Figure 6.51 Time history of the roof maximum interstorey drift of the fixed and isolated six storey RC frames (Cape Mendocino, Butler Valley)

In Figures 6.52-6.57 the maximum drifts between the sequential floors of the six-storey buildings are illustrated. It is observed that the use of seismic isolation highly reduces the drifts of the superstructure, especially in the case of isolators HDNR and in the case of records with 2% probability of exceedance in 50 years. It is also observed that in the case of a record with 2% probability of exceedance the maximum floor acceleration is distributed uniformly along height in the case of D_{LRB} (Figure 6.55). Thus, the base isolation is more active in the case of ground motions with 2% probability of exceedance and especially in the case of Lead Rubber Bearings with reference to maximum floor acceleration.



Figure 6.52 Maximum interstorey drifts of the fixed and isolated six storey RC frames for the bin of records of 2% probability of exceedance



Figure 6.53 Maximum interstorey drifts of the fixed and isolated six storey RC frames for the bin of records of 10% probability of exceedance



Figure 6.54 Maximum interstorey drifts of the fixed and isolated six storey RC frames for the bin of records of 50% probability of exceedance



Figure 6.55 Maximum floor acceleration of the fixed and isolated six storey RC frames for the bin of records of 2% probability of exceedance



Figure 6.56 Maximum floor acceleration of the fixed and isolated six storey RC frames for the bin of records of 10% probability of exceedance



Figure 6.57 Maximum floor acceleration of the fixed and isolated six storey RC frames for the bin of records of 50% probability of exceedance

Figure 6.58 depicts the optimum designs obtained with reference to the type of foundation (fixed, HDNR, LRB), along with the life cycle cost components calculated for the Ghobarah drift limits. A general observation can be obtained from this figure that design without base isolation is worst compared to the other two designs with respect to the life cycle cost (C_{LC}). Comparing design D_{HDNR} with the design D_{fixed} , it can be seen that it is 53% cheaper while it is 26% cheaper compared to the design D_{LRB} with reference to the life cycle cost. Also, design D_{LRB} is cheaper 37% compared to the design D_{fixed} with reference to C_{LC} .



Figure 6.58 Six storey test example - Contribution of the initial cost and life cycle cost components to the total cost for different types of foundation

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Figure 6.59 Six storey test example - Total cost components for different types of foundation

The contribution of the initial and life cycle cost components to the total life-cycle cost are shown in Figure 6.59 .The initial cost (C_{in}) represents the 55% of the total cost for design D_{fixed} while for designs D_{LRB} and D_{HDRN} represents the 71%, 76% respectively. Although the initial cost is the dominant contributor for all optimum designs, in the case of the life cycle cost components the dominant contributor for all designs also is the loss of contents due to floor acceleration, while damage/repair cost represents the second dominant contributor with reference to the life cycle cost components. In particular the loss of contents components is six times that of the repair cost. Worth mentioning also that the loss of contents contribution due to the maximum interstorey drift is insignificant compared to the contribution due to the floor acceleration. Although the three designs differ significantly, injury (minor/major) and fatality costs represent only a small quantity of the total cost: 0.027% for design D_{fixed} , while for designs D_{LRB} and D_{HDRN} represents the 0.0005%, 0.0035% of the total cost, respectively. Thus, all three designs satisfy the life safety criterion.

Comparing design D_{HDNR} with the design D_{fixed} , it can be seen that it is 35% cheaper while it is 4% cheaper compared to the design D_{LRB} with reference to the initial cost (C_{in}). Also, design D_{LRB} is cheaper 31% compared to the design D_{fixed} with reference to C_{in} .

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CHAPTER 7

CONCLUSIONS

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7.1 CONCLUSIONS

For the purposes of this study two test examples are considered. In particular a three storey and a six storey 3D symmetrical RC buildings are used. For each building performance-based optimization problems are formulated and the optimum designs obtained through the different formulations are critically assessed. More specifically both test examples are optimally designed considering fixed support conditions as well as with base isolation. In the latter case Lead Rubber Bearings (LRB) and High-Damping Natural Rubber Bearings (HDNR) isolation systems are examined. For the critical assessment the life-cycle cost analysis procedure is implemented.

Comparing the results of the analysis of the isolated models to the fixed model, significant decrease in the maximum interstorey drift and floor accelerations are observed. As far as the maximum interstorey drift concerned, the reduction observed in the design with Lead-Rubber Bearings is almost the same with the one observed in the design with High Damping Natural Rubbers, compared to the fixed design. Also it is worth mentioning that in the case of the design with Lead-Rubber Bearings and the design with High Damping Natural Rubbers the interstorey drift presents a uniform distribution trend along the floors in the contrary to the design with fixed support conditions, where the interstorey drift presents a reduced trend from the bottom to the roof. Thus, the superstructure of the designs with base isolation moves along the horizontal directions as a rigid body.

Furthermore, the reduction of the maximum accelerations of the design with Lead-Rubber Bearings and the design with High Damping Natural Rubbers is about the same, compared to fixed design. It is noticed that the two isolated structures present a more flexible response where the frequency of the acceleration motion is lower compared to this of the fixed structure. Clearly, reduced accelerations provide significant reduction in the seismic design forces, hence reduce the risk of structural and non-structural earthquake damage.

About the economic assessment of the three designs in both test examples the design with High Damping Natural Rubbers is the most expensive with the design with Lead-Rubber Bearings following with insignificant lower initial cost. The fixed design is defined as the cheapest with reference to the initial cost since the construction of a base isolation is critically cost effective.

With reference to the life cycle cost the design without base isolation is worst compared to the other two designs. Based on the comparison among the three types of design a general observation that the designs with base isolation are cheaper than the the fixed one, along with the fact that the design with High Damping Natural Rubbers is the cheapest. When it comes to the life cycle cost components the dominant contributor for all designs is the loss of contents due to floor acceleration, while damage/repair cost

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represents the second dominant contributor. In particular the loss of contents components is four to six times that of the damage cost. Worth mentioning also that the loss of contents contribution due to the maximum interstorey drift is insignificant compared to the contribution due to the floor acceleration. Although the three designs differ significantly in both test cases, injury (minor/major) and fatality costs represent only a small quantity of the total cost.