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SCHOOL OF GRADUATE STUDIES

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

ASSESSMENT AND RETROFITING MECHANISMS FOR EXISTING REINFORCED CONCRETE BUILDINGS

By

MESFIN GIZAW SHIBESHI

Addis Ababa, Ethiopia | November 2017

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November, 2017

CERTIFICATION

The undersigned have examined the thesis entitled **Assessment and Retrofitting Mechanisms of Existing Reinforced Concrete Buildings** presented by **Mesfin Gizaw Shibeshi,** a candidate for the degree of **Master of Engineering in Structural Engineering** and hereby certify that it is worthy of acceptance.

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Chair person	Signature	Date

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Abstract

In the past many cities around the world have been struck by a range of earthquakes which have resulted in the loss of many lives along with the destruction of properties. Cites have been turned into dust and rubbles. Earthquake causes different intensities of shaking at different locations and the damage induced in buildings at these locations is also different. In many countries, tools have been designed and incorporated in their national building codes to decrease the hazards caused by earthquakes starting from the inception of design to the final stage of construction to harmonize designs for the reduction of seismic risk.

The goal of this study is to focus on the solution mechanisms to be addressed in strengthening of older and existing reinforced concrete framed building to ensure an appropriate level of safety for occupants in the event of an earthquake. As it is known, the earthquake design parameters have been revised in the new Ethiopian Building Code Standard. According to EBCS 8: 1995, Addis Ababa was categorized in zone 2 in the seismic hazard map. But the new revised code locates the city in zone 3, which doubles the peak ground acceleration. More over the city is expanding towards the outskirts of Addis Ababa like Akaki, Bole Arabssa.

In this study, it is considered that the building stock designed earlier before the revision of the code will not survive an earthquake based on the new peak ground acceleration. This brings a question on how these buildings will be strengthened before this kind of destruction and loss of life and property occurs in the city.

Therefore, the outcome of this study will benefit the owners as well as the authorities on the retrofitting procedures as well as solution mechanisms of reinforced concrete framed building structures are undertaken.

Key words-Seismic risk; Serviceability, Seismic hazard; Peak ground acceleration, Retrofitting procedures, Seismic vulnerability; EBCS-8:1995

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1.0 Introduction

Human and economic losses due to earthquake hazards in the past has increased concerns to assess the performance levels of buildings in order to reduce these loses. The seismic hazard and the vulnerability of structures cause risk action [1]. Both of these risk components have inherent uncertainties having their origin in the characteristics of seismic Action [2].

Reinforced concrete structure is widely spread in the building stock all over the world, especially in the seismic areas. This is due to the advantages that these structural systems offer, such as flexibility, easy execution and so on. Many current seismic codes and guidelines include provisions for nonlinear analysis [3-5], which seems to be the natural choice for the evaluation of the behavior of existing structures subjected to design earthquakes.

In Ethiopia's case,' many of the existing buildings in the main cities were built accounting to the seismic actions in EBCS 8: 1995. These reinforced concrete buildings in the country have been designed satisfying the existing building standards and codes at that time. With respect to seismic resistance of the structures, the codes have been followed but the question is that, little is known about their performance when they are actually subjected to unexpected earthquake. With the revision of the building standard and code, the seismic zonings of cities have changed. And with this revision, the ratio of the design bedrock acceleration to the acceleration of gravity is doubled and is 0.1 as compared to 0.05 of EBCS 8, 1995. This fact raises a question on how and what kind of solution mechanisms should be adopted for existing buildings which are going to be damaged or collapsed for an earthquake that have an intensity greater that these structures are designed for.

With the above concerns, this project is conducted with the assumption that buildings that are designed and constructed using older codes needs to be seismically improved, without undergoing the required evaluation techniques for reaching this decision is specifically prescribed.

1.1 Background

The major earthquake threat to human life and property comes from existing substandard buildings, the emphasis of earthquake engineering research, practice and code-writing, on new construction. Policy makers hope that the problem of existing buildings will be solved gradually by wear and tear and sometimes accelerated by urban renewal and redevelopment.

Although seismic resistance adds very little to the construction cost of a new building, the cost of seismic upgrading an existing one, including disruption of use, relocation of tenants, removal and replacement of non-structural parts, etc., is normally a large fraction of the building replacement cost and may be prohibitive for private owners or difficult for the local economy to bear.

Regardless of the economics, seismic retrofitting of buildings is effective in mitigating the seismic risk posed by a substandard building stock. The owner, private or public, may take the initiative individually, often on the occasion of a change in use, architectural remodeling or repair of damage due to an earthquake. Besides, in the context of a broader strategy for seismic risk mitigation, national or local authorities may launch “active” or “passive” seismic assessment and retrofitting programs:

In “active” programs owners of certain categories of buildings may be required to complete the seismic assessment and depending on its outcome, the retrofitting by certain deadlines. The buildings to be targeted may depend:

a) on the seismicity and ground conditions,
b) on the importance, occupancy and perceived vulnerability of the building, as influenced by the type of material and structural system, the number of story, the time of construction with respect to certain benchmark dates of code enforcement, etc.

In “passive” programs seismic assessment, possibly leading to retrofitting is triggered by events or activities related to the use of the building, such as a change that increases occupancy or importance class, repair of damage after an earthquake, remodeling of a minimum percentage of the floor area or above a certain budget, etc.

The seismic performance requirements to be met by the building as is or after retrofitting, if necessary, may be less stringent in compulsory, “active” programs than in “passive” ones. In a “passive” program triggered by remodeling, performance requirements may change with the extent and cost of remodeling works.

The need to retrofit a specific building and the scope and targets of the retrofitting (in terms of weaknesses and deficiencies to be corrected) normally come out of a detailed seismic assessment (or evaluation) of the building. Collapse prevention and the reduction of the risk to life safety for occupants have been the primary goals in the past; for most "Voluntary retrofits. But nowadays, life safety is the primary objective motivation for evaluation and retrofit.

A technically sound seismic assessment is a challenge. However, even when the need to retrofit is obvious, a detailed seismic assessment is worth carrying out: once a structural model of the building as-is set up and analyzed, it can be used at little extra cost as the basis for studying various retrofitting options and for detailed retrofit design. Besides, a detailed assessment in principle provides an objective picture of the seismic vulnerability and resistance, independent of any preconceptions of the engineer doing it.

1.2 Problem Statement

The past 50 or so years have witnessed a dramatic increase in urbanization in Ethiopia associated with increased population. This increase in the urban population has been accompanied by an equally strong growth in the number of high-rise buildings, residential houses, schools, bridges, water supply pipes, and other infrastructure constructions.

Therefore, the proximity of significant earthquakes to the major population centers such as Addis Ababa, Hawassa and Nazret, obviously leads to the question of how much damage will be sustained by these buildings and what kind of solution mechanisms are available to strengthen these buildings for an anticipated earthquake action.

1.3 Motivation

The majorities of buildings in the country, as in many cities in developing countries, were not designed according to strict earthquake design guidelines, and could, unfortunately, sustain significant damage varying from total collapse to structural failure that will render them unfit for occupation under severe earthquake. An increasing number of buildings, especially those built using the previous code, do not even go through a proper quality control procedure to assure compliance with some basic requirements of the country's building code standards. As a result, shabby constructions are increasingly becoming a common practice.

New building codes are primarily intended to regulate the design and construction of new buildings; as such, they include many provisions that encourage the development of designs with features important for good seismic performance, including regular configuration, structural continuity, ductile detailing, and materials of appropriate quality. Many existing buildings were designed and constructed without these features and contain characteristics such as unfavorable configuration and poor detailing that preclude application of building code provisions for their seismic rehabilitation [13].

Concrete buildings will experience, in addition to cracks and foundation failures, beam- column joints failures due to poor reinforcement detailing that could keep them as one unit.

1.4 Objective

The objective of this work is to:

a) Select the type, technique, extent of the intervention required in retrofitting of a building.
b) Provide redesign procedure for structural intervention
c) Provide an analysis and design methodology for use in the retrofit of existing reinforced concrete buildings.
d) Present an overview of the processes used to develop a retrofit strategy and preliminary retrofit design for a building

1.5 Limitations

This project precludes the consideration of the following:

- large number of building population
- retrofitting of foundation structures

1.6 Application of results

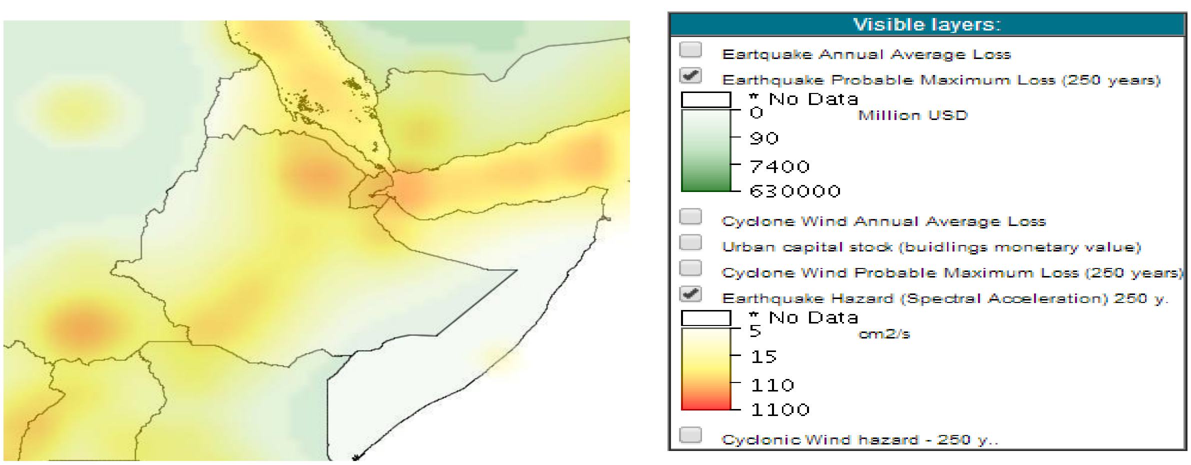
The procedures and techniques of rehabilitation discussed and results obtained from this study can be used as a starting platform for the rehabilitation of existing reinforced concrete framed buildings designed and constructed with the previous EBCS 8: 1995. The discussions in the paper gives the various alternative techniques the can be utilized in retrofitting to make the kind of corrective majors for buildings to continue to give life safety and service for the designed purpose.

2.0 Conceptual Review

2.1 Earthquake Risks in Addis Ababa

According to a report published in 1999, a 6.5 magnitude earthquake, which seismologists say could happen in areas of close proximity to Addis Ababa, the country's major city, could cause as many as 4000-5000 deaths, 8000-10,000 injuries and a displacement of as many as 500,000 people and a total damage in excess of 12 Billion Birr. Addis Ababa itself is only 75-100 kilometers away from the western edge of the Main Ethiopian Rift Valley, which is a hotbed of tremors and active volcanoes. Some of Ethiopia's major cities like Addis Ababa, Nazret, Dire Dawa and Awassa are very near main fault lines such as the Wonji fault, the Nazret fault, the **Addis-Ambo-Ghedo** fault, and the **Fil Woha** fault lines along which numerous earthquakes of varying magnitude have occurred over the years. Other cities like Arba Minch, Dessie, and Mekele are also located in some of the most seismically active areas in the country. The presence of the Fil Woha hot springs in the middle of Addis Ababa itself, for example, is nature's reminder that the city lies on fault lines that have been slowly building strains. It is the release of these strains accumulated over the years that cause the phenomenon of earthquake [6].

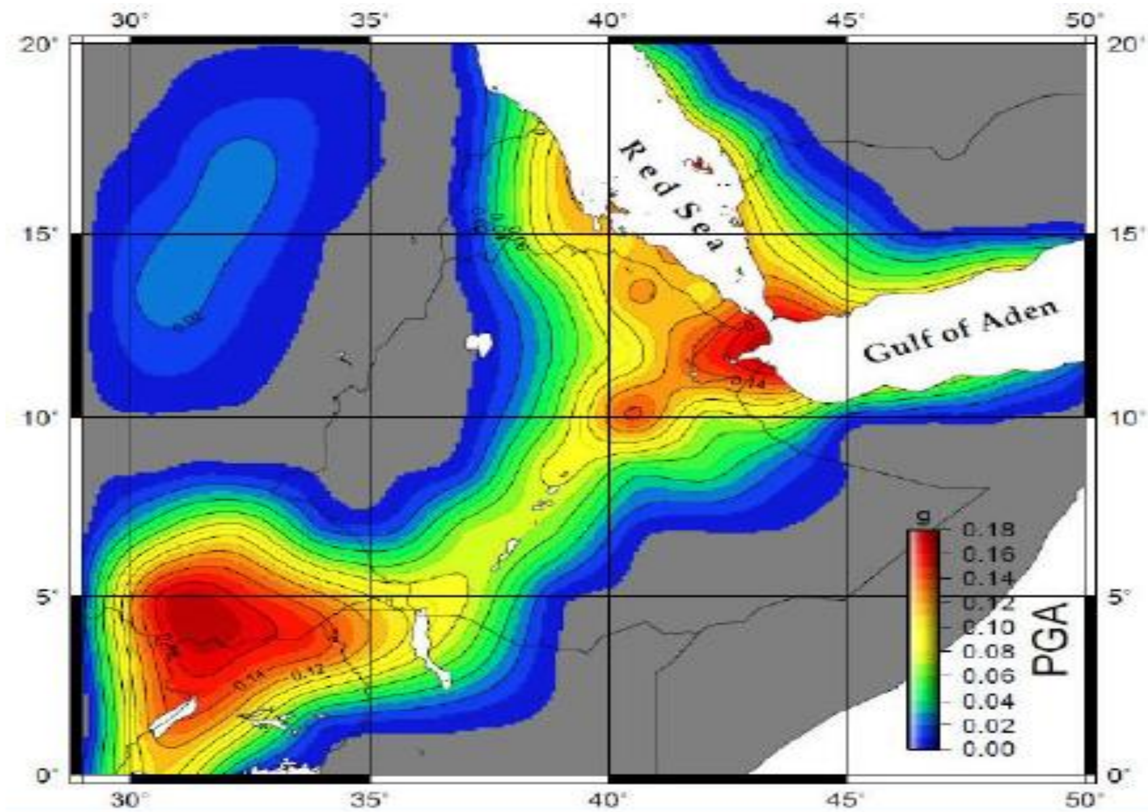
Figure 1 Earthquake risk for Ethiopia and surrounding countries



Source: Map generated from the Global Assessment Report on Disaster Risk Reduction 2013, Risk Data Viewer on 17 January 2014, http://risk.preventionweb.net

According to Pierre Gouin, the earthquakes of 1906 and 1961 shook Addis Ababa and caused widespread panic. Gouin writes, "the magnitude of the main shock of 25 August 1906 was 6.75; the estimated epicentral location, 100 km south of Addis Ababa. The August tremors were exceptionally violent. It is also reported that the shock of 28 October was strong enough to cause the bells of the Church in the Finfine district to ring spontaneously." He further adds, "The population of Addis Ababa was greatly afraid. Damage, however, was slight because: the town, being barely 10-years-old, had not yet fully developed." [8].

Figure 2 Seismic hazard map along the horn of Africa

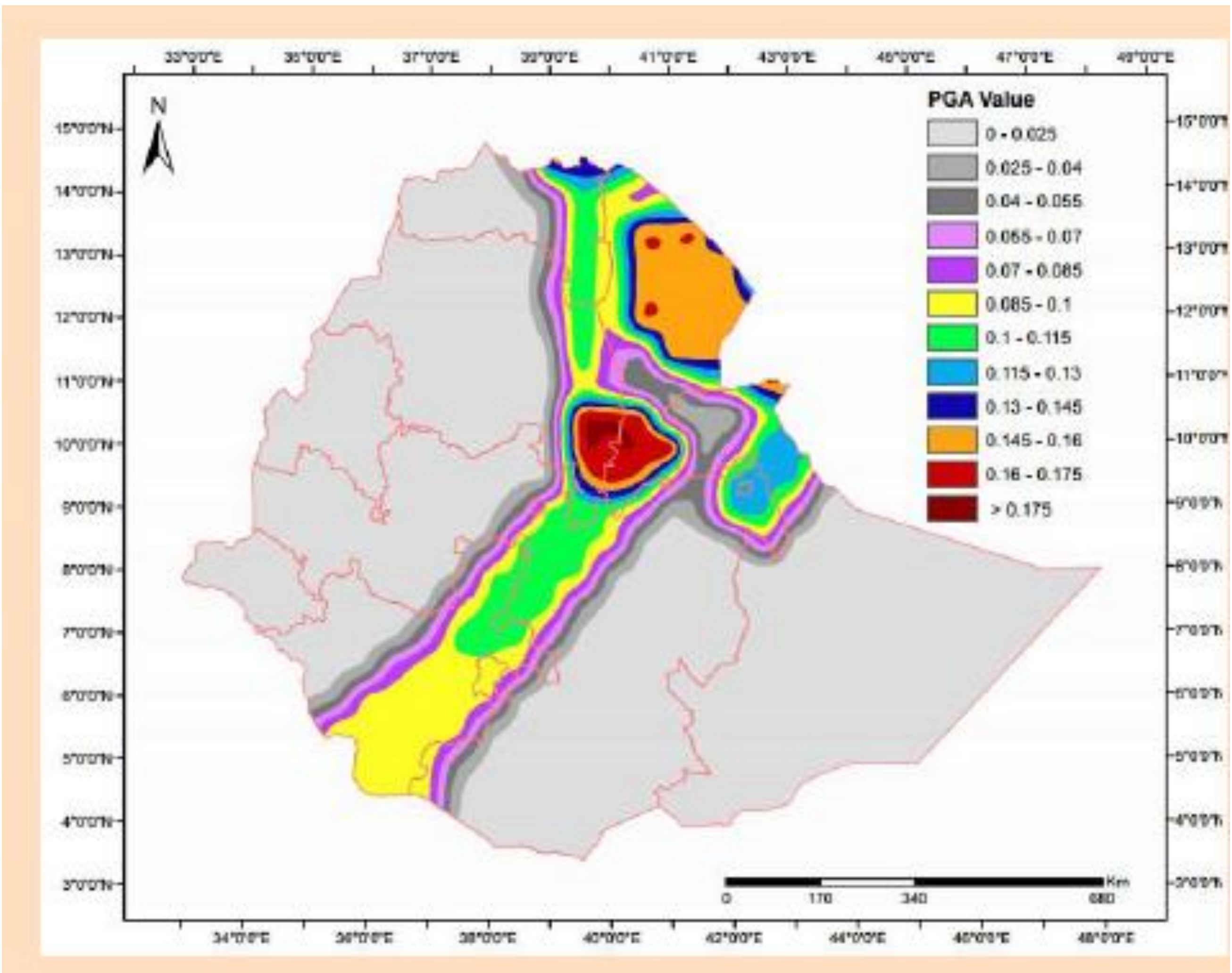


Source: EBCS-EN 1998-1:2014

The pressure from the staggering population growth of the country being a primary one, continue to force rapid implementation of large-scale engineering infrastructure works such as building of mass-housing, water-supply reservoirs, power-plants, dams, new cities, etc. The country's population is projected to reach a staggering 120 million by 2025.

In a recent paper, it has been argued that 25 new cities with the size equivalent to the present Dire Dawa are needed or the current 10 regional cities including Addis Ababa and Dire Dawa will have to become mega cities of 10 million or more to accommodate this growth [9].

Figure 3 Ethiopia's seismic hazard map in terms of peak ground acceleration



Source: EBCS-EN 1998-1:2014

However, a substantial amount of these large infrastructure works already lie or will be in or in close proximity to the some of the most seismically active regions of the country such as Afar Triangle, the Main Ethiopian Rift (MER), and the Southern Most Rift (SMR) where well-documented damage-causing earthquakes are common [9]. A review of the engineering reports associated with some of the largest and most expensive infrastructure projects in the country suggest that - despite the presence of a substantial amount of published literature on the significant seismicity of the region - the severity of threats posed by seismic hazards on the safety and serviceability of these structures is not well-understood by the main stake-holders such as policy-makers, insurance companies, real-estate developers, capital investors, building design-checkers and, the engineering community itself as well.

Based on the figures shown above, the country has been subdivided into seismic zones, depending on the local hazard. The hazard is described in terms of a single parameter, the effective peak ground acceleration, $a_{g,}$ in rock of firm soil, called “design ground acceleration”. As per the new code, Addis Ababa is located in zone of Akaki Kality and categorized in zone 3[1].

Table 1: Design ground acceleration for the new seismic zones as per EBCS-EN 1998-1:2014

Zone	**5**	**4**	**3**	**2**	**1**	**0**
$\boldsymbol{\alpha_0 = a_{g/g}}$	0.2	0.15	0.10	0.07	0.04	0

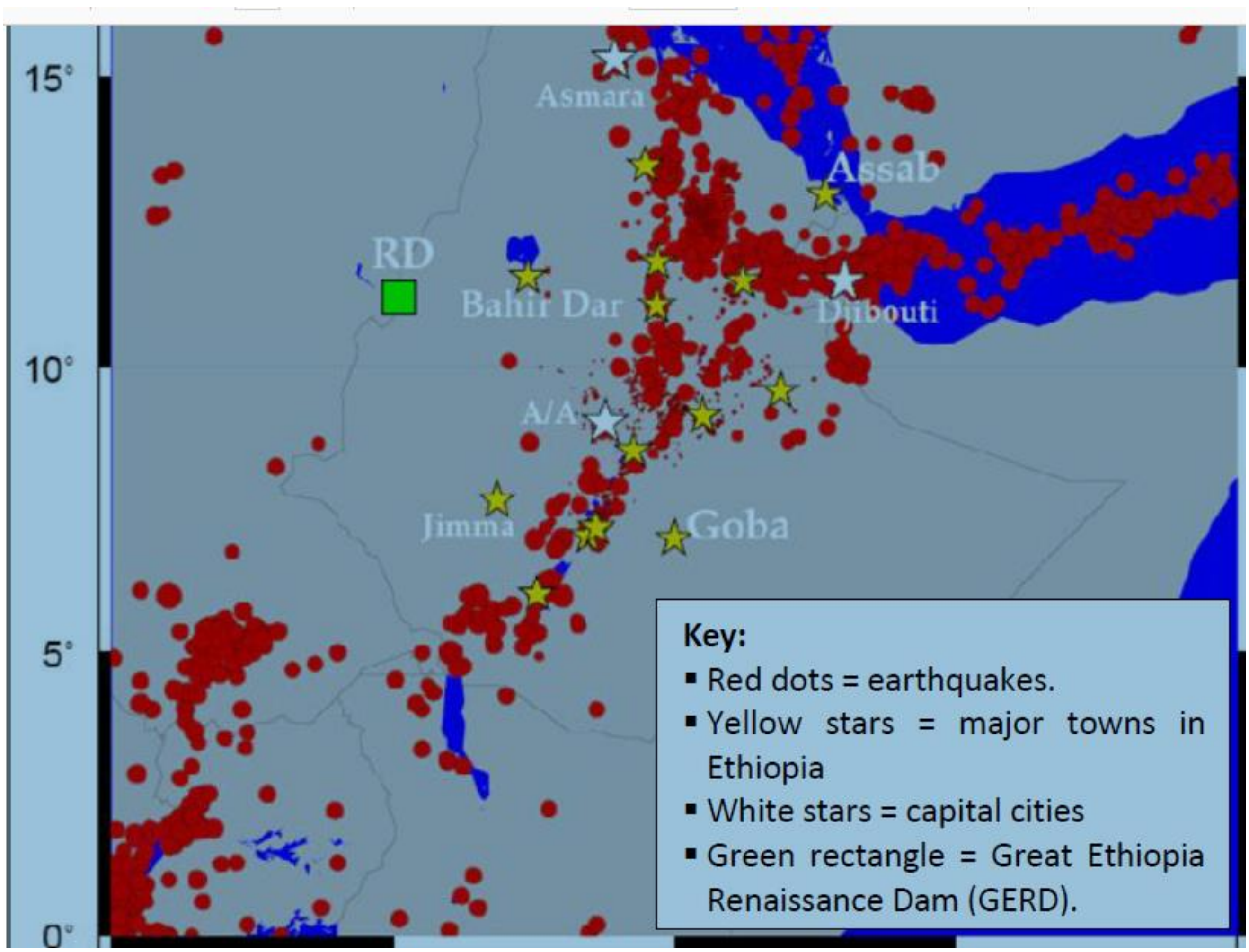
2.2 Review of Response of Built-up Structures to Seismic Events in Ethiopia

While an extensive amount of earthquake records on Ethiopia exist, the structural damage to infrastructures is very low due to the extreme limitation of built-up environments in the country. It is only, perhaps, starting from the 1950s and 1960s one sees what could be characterized as noticeable building and infrastructure activity in the country, particularly in the seismic-prone areas. Therefore, this study concentrates exclusively in the period from 1978 to the present.

For the period between 1960 and 1978, Gouin’s work [8] provides a wealth of information on the response of built-up structures like buildings and bridges to some of the large and damaging earthquakes such as Karakore (1961) and Serdo (1969). Interestingly, this period coincides with a growth in built-up areas and infrastructure in some of the seismically active areas, particularly MER and the Afar Triangle.

Areas where there were no infrastructure damages even under strong ground motions - such as the 6.3 intensity Chabbi Volcano earthquake of 1960 near the present day Awassa - have now seen encroachment of built-up areas which have suffered damages under recent but much less-strong ground motions. Therefore, it has increasingly become clear that structural damages to buildings and infrastructure due to earthquakes are on the rise in the country [9].

Figure 4: Earthquakes recorded in the Horn of Africa region from 1900 to 2010



- http://www.emdat.be/result-country-rofile?disgroup=natural&country=eth&period=1900$2013

2.3 Factors causing Building Distress

Building distress can be as a result of the following factors [17]:

(i) Lack of maintenance of the building which results in deterioration or aging of materials and structural components leading to corrosion and cracking.

(ii) Buildings or structures are damaged at different grades of damage when they experience extreme loading conditions like in severe earthquakes or cyclonic storms for which they are not designed.

(iii) They may also fail if the building including the foundation is not properly designed and constructed following the standard Codes of practice. An impression exists that taller structures are seismically unsafe in comparison with low-rise buildings. On the contrary, when properly designed and built, taller structures are generally safer. It is to be noted that most lives were lost in Kachchh (Gujarat) earthquake of 2001 in one and two-story masonry buildings. Hence, *all* buildings have to be built safe.

(iv) Inadequacy of design and poor quality of construction and maintenance

are therefore the main reasons for the distress seen in buildings during service or under natural hazards. This is because building codes and byelaws are not conscientiously followed in design and quality of construction, nor in maintenance.

3. 0 Seismic Retrofitting/Rehabilitation Assessment Process

Prior to embarking on a rehabilitation program, seismic deficiencies shall have been identified through a prior seismic evaluation or assessment performed using an evaluation methodology considering a combination of building performance and seismic hazard to determine whether the building, in its existing condition, has the desired seismic performance capability [4].

Assessment is a quantitative procedure by which it is checked whether an existing undamaged or damaged building can resist the design seismic load combination. Assessment is made for individual buildings, in order to decide about the need for structural intervention and about strengthening or repair measures to be implemented [2].

There are various strategies and systems that may be applicable to a seismic upgrade project and for selecting the strategy and system that provide an optimal design solution, given the desired performance objectives, building characteristics and technical features of the existing structure.

3.1 Condition Assessment of Existing Building

A condition assessment and site conditions of existing buildings shall be performed as part of the seismic rehabilitation process. The goal of this assessment is:

- To examine the physical condition of the primary and secondary components
- To examine the presence of degradation
- To verify the presence and configuration of components and their connections
- To verify the continuity of load paths between components, elements and systems

The physical condition of existing components and elements, and their connections, must be examined for the presence of degradation. Degradation may include environmental effects (e.g., corrosion, fire damage, chemical attack), or past or current loading effects (e.g., overload, damage from past earthquakes, fatigue, fracture).

The condition assessment shall also examine for configurational problems observed in recent earthquakes, including:

- effects of discontinuous components,
- construction deficiencies,
- poor fit-up, and
- ductility problems.

The main objective of condition assessment is to place the building into one of the following three categories [17]:

- The building has not shown any signs of distress and it satisfies all the safety and serviceability requirements according to relevant Codes of practice, hence no action is needed towards retrofitting.
- The building is seen to be deficient or distressed but it can be repaired and strengthened to satisfy the safety requirements or performance criteria set by codes or the user.
- The building is badly damaged. It is to be demolished and a new building may be built or build back better.

Main steps of condition assessment will be [17]:

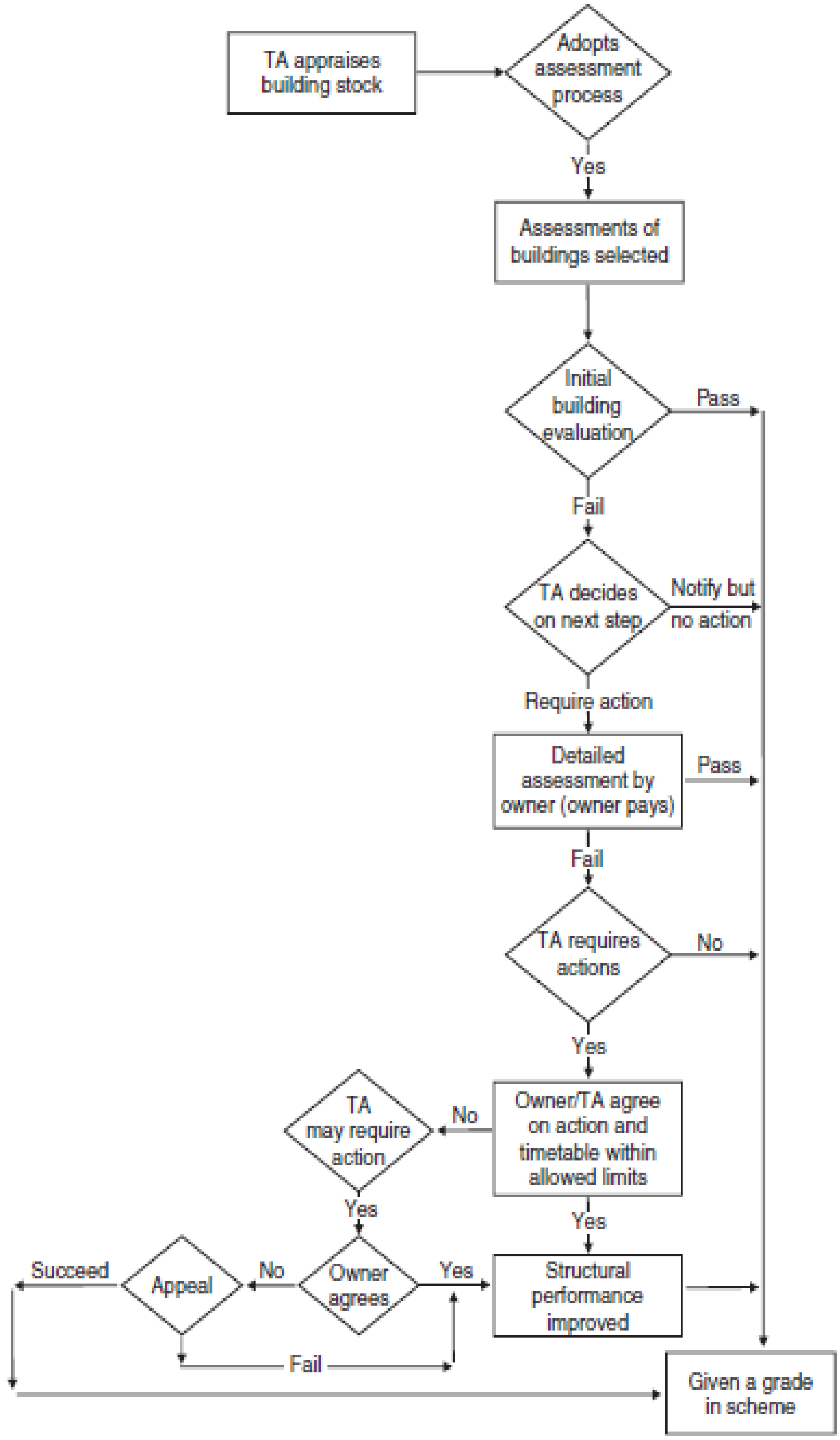
a) To record the damage if any, and find out the causes for distress

b) To assess the extent of distress and to estimate the residual strengths of structural components and the system including the foundation.

c) To plan the rehabilitation and retrofitting or strengthening of the building.

A flowchart for the seismic safety evaluation process of existing buildings is shown in Figure 5. This process is driven by the territorial authority (TA), e.g. city or district council, which has responsibility for local construction standards. The process comprises two phases, an initial TA-funded appraisal followed if necessary by a detailed assessment paid for by the building's owner.

Figure 5: Flowchart for identifying, assessing and retrofitting of buildings, as proposed for New Zealand, TA = Territorial Authority (NZSEE, 2006)



3.2 Scope of Condition Assessment

Condition assessment should include all primary structural elements and components involved in gravity and lateral load resistance, as limited by accessibility. The knowledge and insight gained from the condition assessment is invaluable to the understanding of load paths and the ability of components to resist and transfer these loads. The degree of assessment performed also affects the knowledge level gained and type of analysis [15].

3.3 Methodology of Condition Assessment

Condition assessment and evaluation is generally carried out in two levels:

- Preliminary and
- Detailed.

If we get adequate information to assess the safety of the building at the preliminary investigation level, detailed investigation, which involves considerable cost and time, may not be recommended.

Component orientation, verticality, and physical dimensions should be confirmed during assessment. Connections in concrete components, elements, and systems require special considerations and evaluation [15].

3.3.1 Rapid Visual Investigation

There are mainly three components and steps:

- Collection of information and details about the building design, construction, utilization, and maintenance in the past
- Visual inspection of condition at site and recording details of distress
- Evaluation of safety against the provisions in building codes or specified performance criteria

Direct visual inspection provides the most valuable information, as it can be used to quickly identify any configurational issues, and it allows the measurement of component dimensions, and the determination whether degradation is present.

The continuity of load paths may be established through viewing of components and connection condition. From visual inspection, the need for other test methods to quantify the presence and degree of degradation may be established. The dimensions of accessible primary components shall be measured and compared with available design information [15].

3.3.1.1 Information needed for Rapid investigation

One needs a complete record of building design details and drawings, architectural details, construction details and drawings including the specifications of materials used, geotechnical details of the area and foundation particulars, details of any repair or retrofitting done from the time of construction, details of usage of the building including the loads. If the above information is not available, detailed investigations have to be conducted.

3.3.1.2 Details in visual Investigation

The main purpose of visual investigation is to observe and note down all the items of distress or design deficiency and their locations, supported by sketches and drawings. The visual inspection includes [17]:

- Verification of the accuracy of the original drawings or determination of basic building information, if no drawings are available.
- Identification of major alterations not shown on the original construction documents.
- Identification of visible structural damage, such as concrete cracking or spa1ling, and observations on quality of construction
- Identification of potential non-structural falling hazards, including ceilings, partitions, curtain Walls, parapets, fixtures, and other non-structural building elements.
- Observations on the condition of soil and the foundation
- Documentation of existing conditions with photographs at key locations.

3.3.1.3 Additional Testing

The physical condition of components and connectors may also dictate the need for certain destructive and nondestructive test methods. Such methods may be used to determine the degree of damage or presence of contamination, and to improve understanding of the internal condition and quality of the concrete [15].

3.4 Quantifying Results

The results of the condition assessment shall be used in the preparation of building system models in the evaluation of seismic performance. To aid in this effort, the results shall be quantified, with the following specific topics addressed:

- Component section properties and dimensions
- Component configuration and presence of any eccentricities or permanent deformation
- Connection configuration and presence of any eccentricities
- Presence and effect of alterations to the structural system since original construction (e.g., doorways cut into shear walls)
- Interaction of nonstructural components and their involvement in lateral load resistance

3.5 Required input data

In general, the information for structural evaluation should cover the following points:

a) Identification of the structural system and of its compliance with the regularity criteria in codes.

b) Identification of the type of building foundations.

c) Identification of the ground conditions

d) Information about the overall dimensions and cross-sectional properties of the building elements and the mechanical properties and condition of constituent materials.

e) Information about identifiable material defects and inadequate detailing.

f) Information on the seismic design criteria used for the initial design, including the value of the force reduction factor (*q*-factor), if applicable.

g) Description of the present and/or the planned use of the building (with identification of its importance category.

h) Re-assessment of variable loads considering the use of the building.

i) Information about the type and extent of previous and present structural damages, if any, including earlier repair measures.

Depending on the amount and quality of the information collected on the points above, different types of analysis and different values of the partial safety factors shall be adopted.

3.5.1 As-Built Information

Existing building characteristics pertinent to its seismic performance; including

- its configuration, and the type,
- detailing,
- material strengths, and
- condition of the various structural and nonstructural elements, including foundations and their interconnections shall be determined.

Existing characteristics of the building and site should be obtained from the following sources, as appropriate:

- Field observation of exposed conditions and configuration
- Available construction documents, engineering analyses, reports, soil borings and test logs, maintenance histories, and manufacturers' literature and test data
- Reference standards and codes from the period of
- Destructive and nondestructive examination and testing of selected building components
- Interviews with building owners, tenants, managers, the original architect and engineer, contractor(s), and
- the local building official

As a minimum, at least one site visit should be performed to obtain detailed information regarding building configuration and condition, site and geotechnical conditions, and any issues related to adjacent structures, and to confirm that the available construction documents are generally representative of existing conditions [15].

3.5.2 Building Configuration

The as-built building configuration information shall include data on the type and arrangement of existing structural elements and components of the gravity- and lateral-load-resisting systems, and the nonstructural components of the building that affect either the stiffness or strength of the structural elements, or the structural load path. The structural elements and components shall be identified and categorized as either primary or secondary [4].

Effective load-resisting elements may include structural elements and nonstructural elements that participate in resisting lateral loads, whether or not they were intended to do so by the original designers. Potential seismic deficiencies in intended and effective load resisting elements may include discontinuities in the load path, weak links, irregularities and inadequate strength and deformation capacities [4].

3.5.3 Component Properties

Meaningful structural analysis of a building's probable seismic behavior and reliable design of rehabilitation measures requires good understanding of [4]:

- the existing components (e.g., beams, columns, diaphragms),
- their interconnection, and
- their material properties (strength, deformability, and toughness).

The strength and deformation capacity of existing components should be computed, based on derived material properties and detailed component knowledge.

Existing component action strengths must be determined for two basic purposes:

- To allow calculation of their ability to deliver load to other elements and components, and
- to allow determination of their capacity to resist forces and deformations.

Component deformation capacity must be calculated to allow validation of overall element and building deformations and their acceptability for the selected Rehabilitation Objectives. In general, component capacities are calculated as “expected values” that account for the mean material strengths as well as the probable effects of strain hardening and/or degradation.

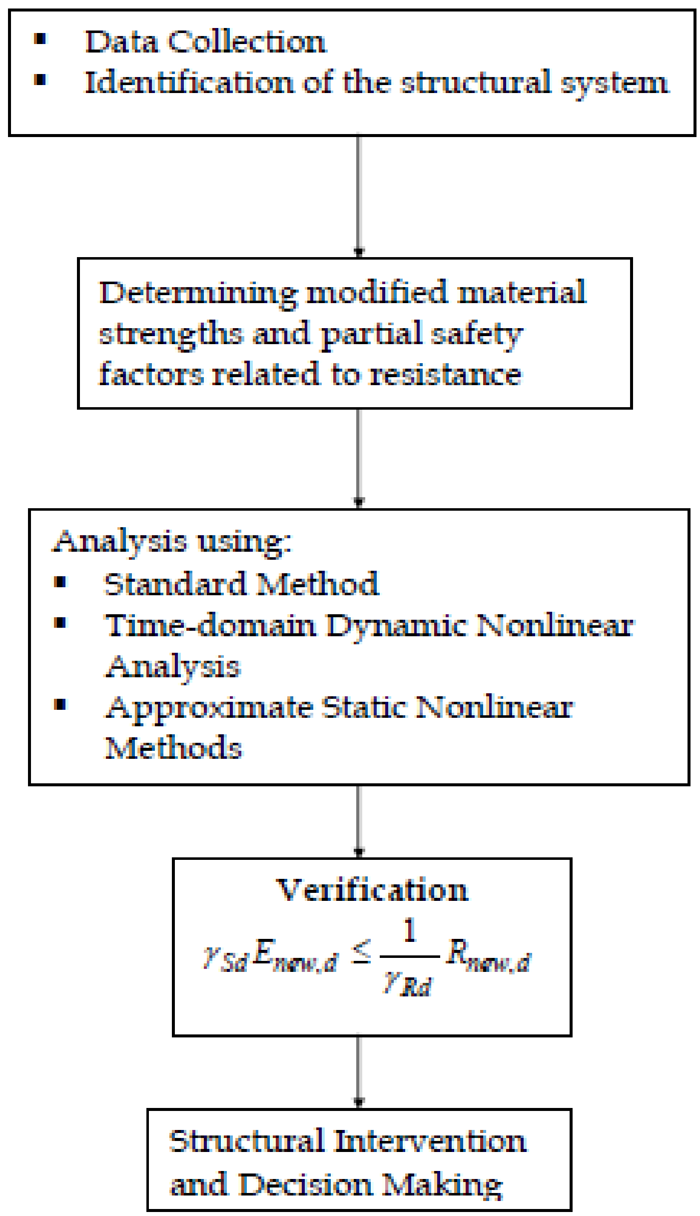


Figure 6: Eurocode 8- Evaluation Procedure (Eurocode, 1996)

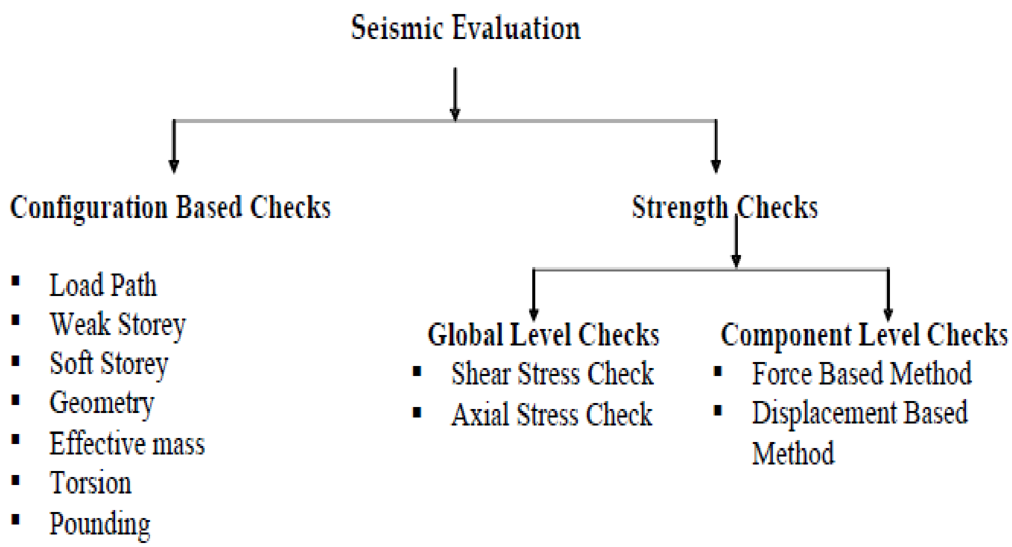


Figure 7: General Structure of Evaluation Procedures

4.0 Knowledge Levels

Since existing structures:

(i) reflect the state of knowledge of the time of their construction,

(ii) possibly contain hidden gross errors,

(iii) may have been submitted to previous earthquakes or other accidental actions with unknown effects,

structural evaluation and possible structural intervention are typically subjected to a different degree of uncertainty or level of knowledge than the design of new structures. Different sets of material and structural safety factors are therefore required, as well as different analysis procedures, depending on the completeness and reliability of the information available [2].

Knowledge of existing component configuration, quality of construction, physical condition, and interconnection to other structural components is necessary to compute strength and deformation capacities. This knowledge should be obtained by:

- visual surveys of condition,
- destructive and nondestructive testing, and
- field measurement of dimensions, as appropriate.

Even with an exhaustive effort to maximize knowledge, uncertainty will remain regarding the validity of computed component strength and deformation capacities. Depending on the amount and quality of information collected on the as-built information, different types of analysis and different values of partial safety factors shall be adopted.

For the purpose of choosing the admissible type of analysis and appropriate partial safety factor values, the following three knowledge levels are defined [2]:

- KL1: Limited Knowledge
- KL2: Normal Knowledge
- KL3: Full Knowledge

4.1 KL1: Limited knowledge

The knowledge level is referred to the following three items:

Geometry: the structure's geometry is known either from survey or from original architectural drawings. In this latter case, a sample visual survey should be performed in order to check that the actual situation of the structure corresponds to the information contained in the drawings and has not changed from the time of construction.

The original architectural drawings are those documents describing the geometry of the structure, allowing for identification of structural components and their dimensions, as well as the structural system to resist both vertical and lateral actions. A visual survey is a procedure for checking correspondence of the actual geometry of the structure with the available original architectural drawings. Sample geometry measurements on selected elements should be carried out.

Details: the structural details are not known from original construction drawings and should be assumed based on simulated design according to usual practice of the time of construction. Limited in-situ inspections in the most critical elements should be performed to check that the assumptions correspond to the actual situation. The information collected should be sufficient to perform local verifications.

Materials: no direct information on the mechanical properties of the construction materials is available, neither from original design specifications nor from original test reports. In this case, default values should be assumed according to standards of the time of construction, accompanied by limited in-situ testing in the most critical elements.

The information collected regards elements dimensions, beams spans and columns heights and is sufficient to build a structural model for linear analysis

4.2 KL2: Normal knowledge

The knowledge level is referred to the following three items:

Geometry: the structure's geometry is known either from survey or from original architectural drawings. In this latter case, a sample visual survey should be performed in order to check that the actual situation of the structure corresponds to the information contained in the drawings and has not changed from the time of construction.

The information collected regards elements dimensions, beams spans and columns heights and is sufficient, together with those regarding the details, to build a structural model for either linear or nonlinear analysis.

Details: the structural details are known either from extended in-situ inspection or from incomplete original executive construction drawings. In the latter case, limited in-situ inspections in the most critical elements should be performed to check that the available information corresponds to the actual situation.

The information collected should be sufficient for either performing local verifications or setting up a nonlinear structural model.

Materials: information on the mechanical properties of the construction materials is available either from extended *in-situ* testing or from original design specifications. In this latter case, limited in-situ testing should be performed.

The information collected should be sufficient for either performing local verifications or setting up a nonlinear structural model. Structural evaluation based on a state of normal knowledge shall be performed through either linear or nonlinear analysis methods, either static or dynamic and the relevant partial safety factors for the material properties shall be taken.

4.3 KL3: Full knowledge

The knowledge level is referred to the following three items:

Geometry: the structure's geometry is known either from survey or from original architectural drawings. In this latter case, a sample visual survey should be performed in order to check that the actual situation of the structure corresponds to the information contained in the drawings and has not changed from the time of construction.

The information collected regards elements dimensions, beams spans and columns heights and is sufficient, together with those regarding the details, to build a structural model for both linear and nonlinear analysis.

Details: the structural details are known either from comprehensive in-situ inspection or from original executive construction drawings. In the latter case, limited in-situ inspections in the most critical elements should be performed to check that the available information corresponds to the actual situation.

The information collected should be sufficient for either performing local verifications or setting up a nonlinear structural model.

Materials: information on the mechanical properties of the construction materials is available either from comprehensive in-situ testing or from original test reports. In this latter case, limited in-situ testing should be performed.
The information collected should be sufficient for either performing local verifications or setting up a nonlinear structural model.

Structural evaluation based on a state of full knowledge shall be performed through either linear or nonlinear analysis methods, either static or dynamic. The relevant partial safety factors for the material properties shall be appropriately decreased.

The following characteristics represent the minimum appropriate level of effort in gaining knowledge of structural configuration:

- Records of the original construction and any modifications, including structural and architectural drawings, and in the absence of structural drawings, a set of record drawings and/or sketches is prepared, documenting both gravity and lateral systems.
- A visual condition survey is performed on the accessible primary elements and components, with verification that the size, location, and connection of these elements is as indicated on the available documentation.
- A limited program of in-place testing is performed, to quantify the material properties, component condition, and dimensions of representative primary elements with quantification of the effects of any observable deterioration.
- Knowledge of any site-related concerns from field survey and research; such as pounding from neighboring structures, party wall effects, and soil or geological problems including risks of liquefaction.
- Specific foundation- and material-related concerns and knowledge of their influence on building performance.

4.4 Comprehensive Knowledge

Comprehensive knowledge may be assumed when all of the following factors exist:

- Original construction records, including drawings and specifications, as well as any post-construction modification data that explicitly depict as-built conditions. Where such documents are not available, drawings and sketches are developed based on detailed surveys of the primary structural elements. Such surveys include destructive and/or nondestructive investigation as required to determine the size, number, placement,
  and type of obscured items such as bolts and reinforcing bars. In addition, documentation is developed for representative secondary elements.
- Extensive in-place testing is performed to quantify material properties, and component conditions and dimensions or records of the results of quality assurance tests constructed during testing are available.

- Knowledge of any site-related concerns—such as pounding from neighboring structures, party wall effects, and soil or geological problems including thorough visual survey and research efforts.
- Specific foundation- and material-related concerns have been examined and knowledge of their influence on building performance has been gained.

4.5 Performance Levels

Building performance is expressed in terms of building performance levels. Building Performance Levels are discrete damage states selected from among the infinite spectrum of possible damage states that buildings could experience as a result of earthquake response. These Performance Levels have readily identifiable consequences associated with the post-earthquake disposition of the building that are meaningful to the building user community. These include the ability to resume normal functions within the building, the advisability of post-earthquake occupancy, and the risk to life safety [19].

The consequences of earthquakes on buildings can be categorized into three types of losses:

- Life Safety: deaths and injuries to building occupants and passersby
- Capital Losses: costs to repair or replace the building or its contents
- Functional Losses: loss of revenue or increase in expenses related to the inability of a facility to function normally after earthquakes

The level of performance for a building during an earthquake is measured by the nature and extent of these potential losses. Obviously, the level of performance is affected by the strength of each earthquake. It is reasonable to expect that a building remains safe, i.e., not cause life loss, for rare large earthquakes and that it remains usable for more-frequent moderate events. A performance objective is a goal that a building achieves a certain level of performance for a specific level of seismic ground shaking hazard. An owner might decide that the goals for a 'building should' be to remain life safe for the Maximum Earthquake hazard level and functional after a Serviceability Earthquake' hazard level event [5].

4.5.1 Structural Performance Levels and Ranges

When a building is subjected to earthquake ground motion, a pattern of lateral deformations that varies with time is induced into the structure. At any given point in time, a particular state of lateral deformation will exist in the structure, and at sometime within the period in which the structure is responding to the ground motion, a maximum pattern of deformation will occur. At relatively low levels of ground motion, the deformations induced within the building will be limited, and the resulting stresses that develop within the structural components will be within the elastic range of behavior. Within this elastic range, the structure will experience no damage. All structural components will retain their original strength, stiffness, and appearance, and when the ground motion stops, the structure will return to its pre-earthquake condition [19].

At more severe levels of ground motion, the lateral deformations induced into the structure will be larger. As these deformations increase, so will demands on the individual structural components. At different levels of deformation, corresponding to different levels of ground motion severity, individual components of the structure will be strained beyond their elastic range. As this occurs, the structure starts to experience damage in the form of cracking, spalling, buckling, and yielding of the various components.

As components become damaged, they degrade in stiffness, and some elements will begin to lose their strength. In general, when a structure has responded to ground motion within this range of behavior, it will not return to its pre-earthquake condition when the ground motion stops. Some permanent deformation may remain within the structure and damage will be evident throughout. Depending on how far the structure has been deformed, and in what pattern, the structure may have lost a significant amount of its original stiffness and, possibly, strength. At higher levels of ground motion, the lateral deformations induced into the structure will strain a number of elements to a point at which the elements behave in a brittle manner or, as a result of the decreased overall stiffness, the structure loses stability. Eventually, partial or total collapse of the structure can occur. Brittle elements are not able to sustain inelastic deformations and will fail suddenly; the consequences may range from local and repairable damage to collapse of the structural system.

Figure 8 below illustrates the behavior of a ductile structure as it responds with increasing lateral deformation. The figure is a schematic plot of the lateral force induced in the structure as a function of lateral deformation. Three discrete points are indicated, representing the discrete Performance Levels: Immediate Occupancy, Life Safety, and Collapse Prevention [19].

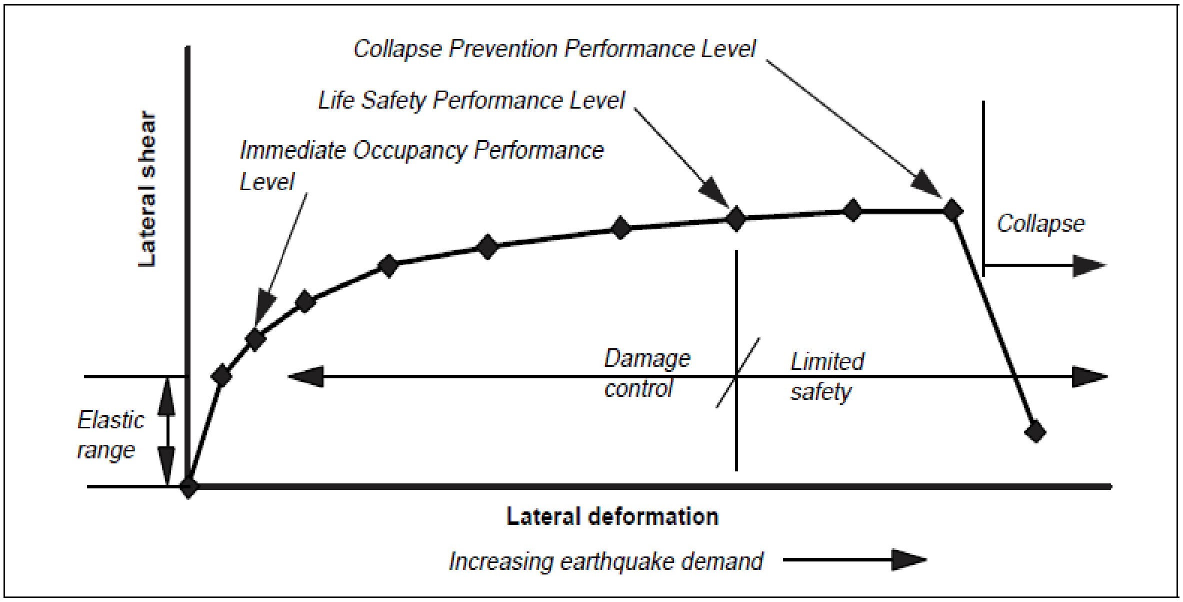


Figure 8: Performance and Structural Deformation Demand for Ductile Structures

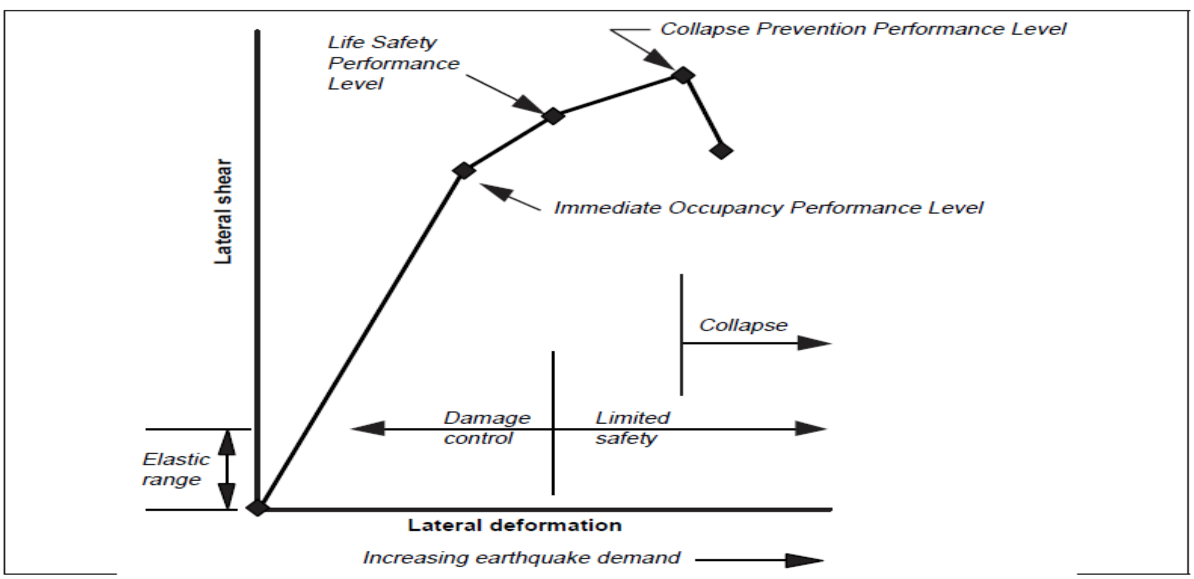


Figure 9: Performance and Structural Performance Demand for Nonductile Structures

At the Immediate Occupancy Level, damage is relatively limited. The structure retains a significant portion of its original stiffness and most if not all of its strength.

At the Collapse Prevention Level, the building has experienced extreme damage. If laterally deformed beyond this point, the structure can experience instability and collapse.

At the Life Safety Level, substantial damage has occurred to the structure, and it may have lost a significant amount of its original stiffness. However, a substantial margin remains for additional lateral deformation before collapse would occur.

Performance Level		Description
NEHRP Guidelines	Vision 2000	
Operational	Fully Functional	No significant damage has occurred to structural and non-structural components. Building is suitable for normal intended occupancy and use.
Immediate Occupancy	Operational	No significant damage has occurred to structure, which retains nearly all its pre-earthquake strength and stiffness. Nonstructural elements are secure and most would function, if utilities were available. Building may be used for intended purpose, albeit in an impaired mode.
Life Safety	Life Safe	Significant damage to structural elements with substantial reduction in stiffness, however margin remains against collapse. Nonstructural elements are secured but may not function. Occupancy may be prevented until repairs can be instituted.
Collapse Prevention	Near Collapse	Substantial structural and nonstructural damage. Structural strength and stiffness substantially degraded. Little margin against collapse. Some falling debris hazards may have occurred.

Table 2: Structural Performance levels definition (taken from Antoniou 2002)

For a given structure and design earthquake, it is possible to estimate the overall deformation and force demand on the structure and, therefore, the point on the corresponding curves shown in the figures to which the earthquake will push the building. This either will or will not correspond to the desired level of performance for the structure. When structural/seismic rehabilitation is performed, modifications to the structure are made to alter its strength, stiffness, or ability to dampen or resist induced deformations. These actions will alter the characteristics of both the shape of the curves in these figures and the deformation demand produced by the design earthquake on the building, such that the expected performance at the estimated deformation level for the rehabilitated structure is acceptable [19].

4.6 Seismic action and seismic load combination

The basic models for the definition of the seismic motion are those presented in 3.2.2 and 3.2.3 of EN 1998-1. Reference is made in particular to the elastic response spectrum given in 3.2.2.2 of EN 1998-1, scaled with the values of the design ground acceleration established for the verification of the different Limit States. The design seismic action shall be combined with the other appropriate permanent and variable actions in accordance with the rule given in 3.2.4 of EN 1998-1 [2].

4.7 Partial Safety Factors

Based on the knowledge level achieved through the different levels of survey, inspection and testing, the values of the partial safety factors (PSF) shall be established [2].

Table 3: Recommended values of the partial safety factors (PSF) used for verifications, according to the different knowledge levels (KL)

Knowledge Level	Partial Safety Factors
KL1	1.20 γ_m
KL2	γ_m as in EN 1998 -1
KL3	0.80 γ_m

5.0 Structural modelling

Based on the information collected as indicated a model of the structure shall be set up. The model shall be adequate for determining the action effects in all structural elements under the seismic load combination. All provisions of EN 1998-1 regarding modelling (4.3.1) and accidental torsional effects (4.3.2) apply without modifications

NOTE The diaphragm is taken as being rigid, if, when it is modelled with its actual in-plane flexibility, its horizontal displacements nowhere exceed those resulting from the rigid diaphragm assumption by more than 10% of the corresponding absolute horizontal displacements in the seismic design situation.

In order to account for uncertainties in the location of masses and in the spatial variation of the seismic motion, the calculated center of mass at each floor *i* shall be considered as being displaced from its nominal location in each direction by an accidental eccentricity.

5.1 Methods of analysis

The seismic action effects, to be combined with the effects of the other permanent and variable loads according to the seismic combination in 4.2 of EN 1998-1, may be evaluated using one of the following methods [3]:

- lateral force analysis (linear),
- multi-modal response spectrum analysis (linear),
- non-linear static analysis,
- non-linear time history dynamic analyses.

The seismic action to be used is the one corresponding to the elastic (*i.e.*, unreduced by the behavior factor *q*) response spectrum in 3.2.2.2 of EN 1998-1, or its equivalent alternative representations given in 3.2.3 of EN 1998-1, respectively, factored by the appropriate importance factor γ_I (see 4.2.5 of EN 1998-1).

Buildings are classified in four importance classes, depending on the consequences of collapse for human life, on their importance for public safety and civil protection in the immediate post-earthquake period, and on the social and economic consequences of collapse.

The importance classes are characterized by different importance factors $\gamma_{I.}$ The importance factor γ_I = 1,0 is associated with a seismic event having the reference return period T_{NCR} of the seismic action for the no-collapse requirement (or equivalently the reference probability of exceedance in 50 years

5.1.1 Non-linear static (pushover) analysis

Pushover analysis is a non-linear static analysis carried out under conditions of constant gravity loads and monotonically increasing horizontal loads. It may be applied to verify the structural performance of newly designed and of existing buildings for the following purposes [3]:

- to verify or revise the over strength ratio values αu/α1 (see **5.2.2.2**, **6.3.2**, **7.3.2**);
- to estimate the expected plastic mechanisms and the distribution of damage;
- to assess the structural performance of existing or retrofitted buildings for the purposes of EN 1998-3;

Generally, nonlinear procedures provide a more realistic indication of the demands on individual components of structures that are loaded significantly beyond their elastic range of behavior, than do linear procedures [19].

They are particularly useful in that they provide:

- More realistic estimates of force demand on potentially brittle components (force-controlled actions), such as axial loads on columns and braces
- More realistic estimates of deformation demand for elements that must deform inelastically in order to dissipate energy imparted to the structure by ground motions
- More realistic estimates of the effects of individual component strength and stiffness degradation under large inelastic demands

- More realistic estimates of inter-story drifts that account for strength and stiffness discontinuities that may develop during inelastic response
- Identification of critical regions in which large deformation demands may occur and in which particular care should be taken in detailing for ductile behavior
- Identification of strength discontinuities in plan or elevation that can lead to changes in dynamic characteristics in the inelastic range

When performing a pushover analysis many design factors need to be considered. The expected accelerations and displacements of a structure from a seismic event can be designed given the geographic location of the structure combined with the type of foundation the structure is built upon. Given the natural period of the structure, probabilistic acceleration and displacement demands can be defined. These acceleration and displacement demands can be designed for the given size and geometry of columns in use. The formation of plastic hinges in reinforced concrete can then be designed based on the moment-curvature relationship of the column. As plastic hinges form, the forces within the structure will be redistributed based on the revised stiffness in the structure [21].

In the moment curvature analysis, the strain at the most extreme fiber of compression is increased from the initial strain, due to the constant axial load, to the ultimate compressive strain ϵ_{cu} equal to 0.003 for a particular axial load applied to the cross section. With the combination of the strain and axial load known, the neutral axis can be found, which yields the moment at each particular strain. The curvature can then be defined as the strain divided by the depth of the compression zone. Each strain will yield a specific curvature and corresponding moment which can be plotted to find the plastic moment of section, or the moment required to form a plastic hinge [21].

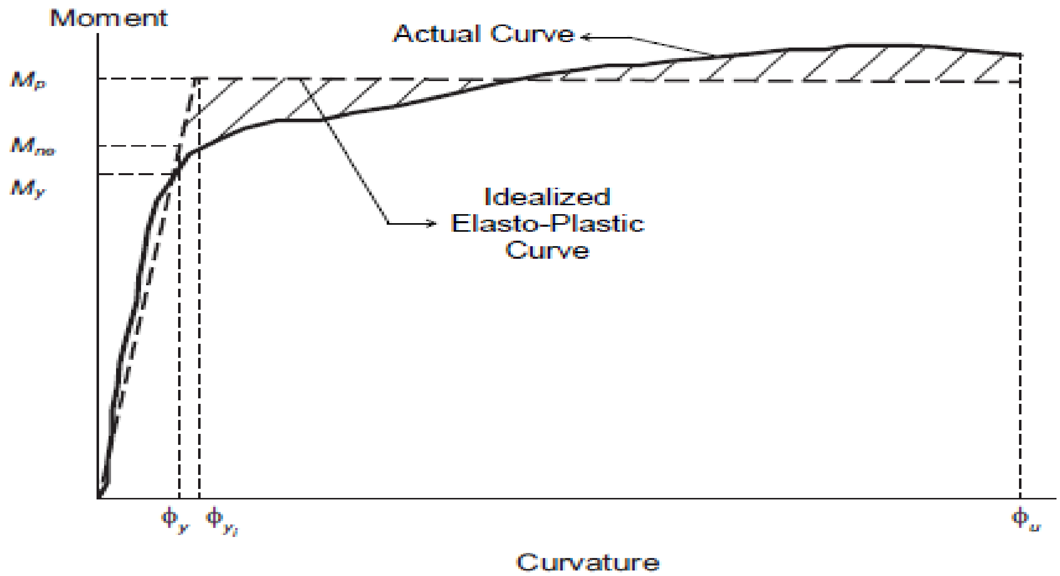


Figure 10: Moment curvature plot for a particular axial load

For the purpose of design, the moment curvature curve shall be idealized with an elastic perfectly plastic response to estimate the plastic moment capacity of a member's cross section. The elastic portion of the idealized curve shall pass through the point at which the first longitudinal reinforcing bar yields. The results from this analysis are used to establish the rotational capacity of plastic hinges as well as the associated plastic deformations. The process of using the moment curvature sectional analysis to determine the lateral load displacement relationship of a frame, column, or pier is defined as a pushover analysis.

The displacement capacity of a structural system for seismic design, given the expected plastic moment and plastic curvature that a column will yield under can be determined. By following a few simple hand calculations, we can obtain the shear force required for the first hinge to form as well as the total displacement demand [21].

5.1.2 Shear Required for Hinge Formation

For a single beam-column element, the shear required for the first hinge can be calculated given the plastic moment values of the specified cross section. Assuming fix-fix boundary conditions, the moment diagram can be defined as shown in figure 11 [21].

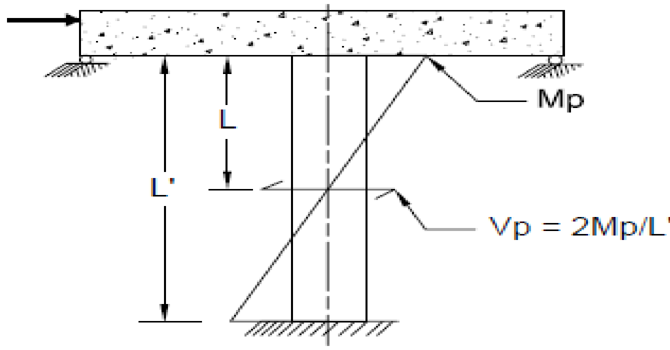


Figure 11: Moment Distribution over height of Pier

Where

$$\text{Vp} = \frac{2M_P}{L'}$$

Static, sequential nonlinear procedure approach avoids many of the inaccuracies inherent in the linear methods by permitting direct, although approximate, evaluation of the inelastic demands produced in the building by the design earthquake. As with the linear procedures, a mathematical model of the building, representing both the existing and new elements, is constructed.

However, instead of performing an elastic analysis of the response of the structural model to specified ground motion, an incremental nonlinear analysis is conducted of the distribution of deformations and stresses throughout the structure as it is subjected to progressively increased lateral displacements. Acceptance criteria include permissible deformation (for example, elongations, drifts, and rotations) and strength demands on common elements and components for different Performance Levels.

By comparing the results of the incremental force displacement analysis ("pushover") with these acceptance criteria, it is possible to estimate limiting overall structural displacements at which each desired Structural Performance Level can be achieved

5.1.3 Displacement Capacity

Individual member displacements are defined as the portion of global displacement attributed to the elastic column idealized displacement Δ_{yi} and plastic displacement demand Δ_{pd} of an equivalent member from the point of maximum moment to the point of contraflexure. AASHTO uses the following figure in determining these values [21].

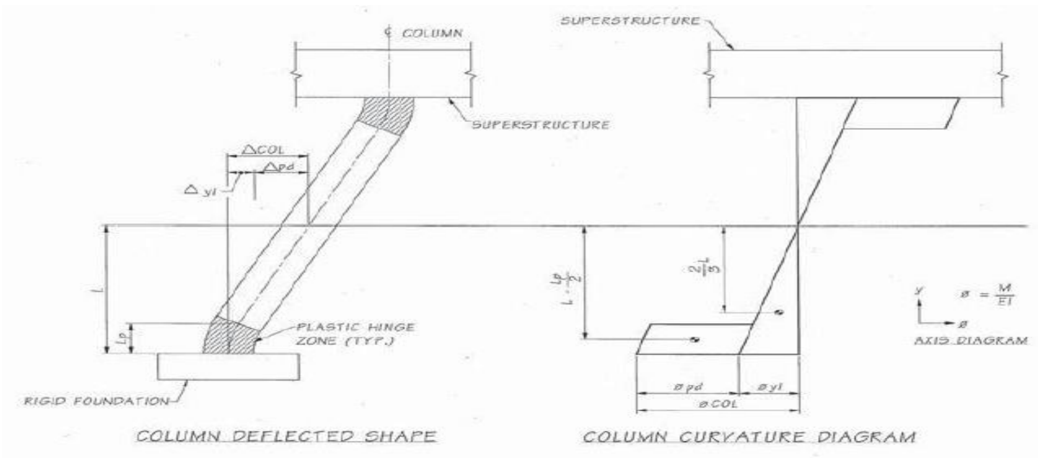


Figure 12: (AASHTO Fig. C4.9-2) Pier-Deflected Shape and Curvature Diagram

Lp is defined as the length of the plastic hinge. For a reinforced concrete column framing into a footing, an integral bent cap, an oversized shaft, or a cased shaft, the plastic hinge length, Lp in inches, can be calculated as

$$L_p = 0.08\,L + 0.15\,f_{ye}\,d_{bl} \geq 0.3 f_{ye}\,d_{bl} \qquad \text{(AASHTO 4.11.6-1) (2.91)}$$

Where L is defined as the length of the column from the point of maximum moment to the point of contraflexure in inches, d_{bl} is the diameter of longitudinal reinforcing steel. From here the elastic displacement can be found as

$$\Delta e = \frac{\phi_{yi}\, L^2}{3} \qquad \text{(AASHTO C4.9-3) (2.92)}$$

Where ϕ_{yi} is the idealized yield curve as shown in Figure 10 and can be solved for as

$$\phi_{yi} = \frac{M_y}{EI}$$

$$M_{yi} = \frac{V_P L}{2}$$

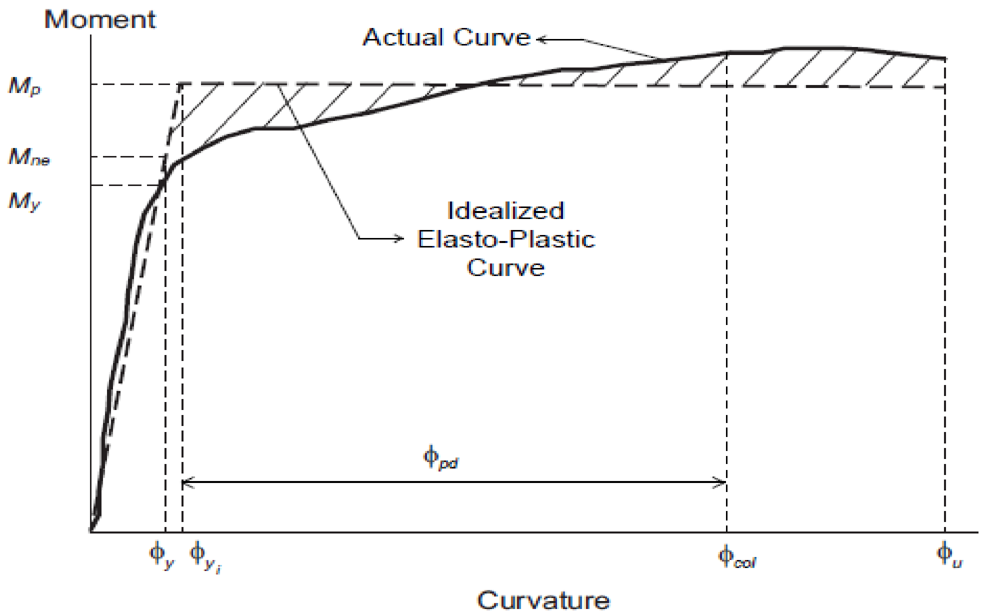


Figure 13: Moment Curvature Diagram with Plastic Curvature Demand

$$\Delta_{pd} = \phi pd\ Lp\ (L - \frac{L}{2}) \qquad \text{(AASHTO C4.9-4)}$$

With the total displacement demand equating to:

$$\Delta_{Tot} = \Delta_{pd} + \Delta_e$$

5.1.4 Capacity Requirements

Checks are to be performed throughout the Pushover analyses presented above which will control the sizing of beam-columns and reinforcement detailing. Additional analyses outside the scope of the pushover analyses may need to be performed in the event that certain parameters are not met, specifically P-delta requirements [21].

5.2 Lateral loads

At least two vertical distributions of lateral loads should be applied:

- a "uniform" pattern, based on lateral forces that are proportional to mass regardless of elevation (uniform response acceleration)
- a "modal" pattern, proportional to lateral forces consistent with the lateral force distribution determined in elastic analysis

Lateral loads shall be applied at the location of the masses in the model and accidental eccentricity should be considered.

5.3 Capacity curve

The relation between base shear force and the control displacement (the "capacity curve") should be determined by pushover analysis for values of the control displacement ranging between zero and the value corresponding to 150% of the target displacement. The target displacement is defined as the seismic demand derived from the elastic response spectrum of 3.2.2.2 of EN 1998-1 in terms of the displacement of an equivalent single-degree-of-freedom system [3].

5.4 Decisions for structural intervention

5.4.1 Criteria for a structural intervention

On the basis of the conclusions of the assessment of the structure and/or the nature and extent of the damage, decisions should be taken, seeking to minimize the cost of intervention and to optimize social interests.

The selection of the type, technique, extent and urgency of the intervention shall be based on the structural information collected during the assessment of the building.

The following aspects should be considered [2]:

- All identified local gross errors should be appropriately remedied.
- In case of highly irregular buildings (both in terms of stiffness and over strength distributions), their structural regularity should be improved as much as possible, both in elevation and in plan.
- The required characteristics of regularity and resistance can be achieved by either direct strengthening of a (reduced) number of deficient components, or by the insertion of new lateral load-resisting elements.
- The increase of local ductility should be sought where needed.
- The increase in strength after the intervention should not reduce the necessary global available ductility.
- Specifically, for masonry structures: non-ductile lintels should be replaced, inadequate connections between floor and walls should be improved, horizontal thrusts against walls should be eliminated.

6.0 Retrofitting of Existing Concrete Buildings

In many parts of the world, design of new buildings for earthquake resistance is a relatively recent development. Resistance of buildings to lateral forces resulted in the past only from wind considerations. Provisions for seismic design and detailing of members and structures resembling those found in modern seismic codes did not appear before the mid-1970s in US standards, or the mid-1980s in European national codes. Although today and for the years to come the major earthquake threat to human life and property comes from existing substandard buildings. Seismic retrofitting denotes upgrading the safety of damaged or existing deficient buildings [11].

6.1 Retrofitting Strategies

Improving the structural performance of buildings in earthquake may be achieved by adopting one or more of the strategies outlined below. Designers are required to carefully consider issues of relative stiffness and relative ductility between the existing structure and new strengthening elements. Strategies include identification of weak or brittle elements that form part of the seismic resisting structure for strengthening. Other strategies involve structural improvements to mitigate poor building global behavior such as soft story mechanisms or highly torsional responses. Ideally, un strengthened and/or strengthened buildings will have an adequate level of redundancy so that localized failure or overload of a few elements will not precipitate overall instability or collapse of the building [20].

The aim of retrofitting is to modify the seismic demands, and/or the capacities, so that all relevant elements of the retrofitted building fulfill the general verification, at all performance levels ("Limit States") under the corresponding seismic action [11].

6.1.1 Strategies for Improving Structural Performance

General strategies for improving structural performance of reinforced buildings can be:

1. By reducing the seismic demands on members and the structure as a whole;
2. By increasing the member capacities.

The most common strategies adopted in the field of seismic retrofit of existing structures are:

- the restriction or change of use of the building,
- partial demolition and/or mass reduction,
- removal or lessening of existing irregularities and discontinuities,
- addition of new lateral load resistance systems,
- local or global modification of elements and systems.

The above overview of the rehabilitation strategies shows that the structural performances of an existing building can be enhanced in different ways by acting on [11]:

- ductility,
- stiffness or
- strength (separately or, in many cases, at the same time);

in each case, a preliminary assessment of the existing structure performances and the evaluation of the analysis results are necessary to select the rehabilitation technique that meets the required performance targets.

The choice of the technique depends on:

a. the locally available materials and technologies;
b. cost considerations;
c. the disruption of use it entails and the duration of the works;
d. architectural, functional and aesthetic considerations or restrictions, etc.

One or more retrofitting technique(s) are normally chosen on the basis of considerations (a–d), etc. This choice and how it is implemented determines which retrofitting strategy is being adopted. Each retrofitting intervention is a special case, with more than one appropriate solutions. So, generalization of rules is neither possible nor advisable.

There are some general (but not absolute) guidelines to follow, depending on the outcome of the assessment of the as-built structure [11]:

1. If there is general deficiency in the building, retrofitting Strategy no. 1 above is more cost-effective, as it can reduce the seismic demands throughout.
2. If there are capacity deficiencies in just a few scattered members, it is more cost effective to focus on them and upgrade their capacities with retrofitting Strategy no. 2.
3. If the deficiencies are concentrated in a single or few (“weak”) story, they may be due to a vertical irregularity. Retrofitting Strategy no. 2 is an option, to upgrade the capacities of the members of these stories. Retrofitting Strategy no. 1 could be adopted instead, to remove the irregularity by adding strong and stiff new elements from the ground to the weak story(s) and beyond, or to strengthen and stiffen existing elements there to overshadow the irregularity and suppress story-sway mechanisms.
4. If the deficiencies are concentrated at a single side of the building, they may be due to a torsional imbalance in plan. It may be chosen to stiffen and strengthen existing elements on that side or add new ones there, to balance the stiffness and strength (retrofitting Strategy no. 1). Alternatively, the deformation capacity and the shear strength of the members of the “flexible side” may be upgraded, to accommodate the larger demands on them (retrofitting Strategy no. 2).

Seismic strengthening for improved performance in the future earthquakes can be achieved by one of several options such as:

a. Local intervention methods and
b. Global intervention methods

6.2 Local Modification of Components

While some existing buildings have substantial strength and stiffness, often some of their structural components are understrength or they have inadequate deformation capacity. A strategy for this type of building, could involve local improvements to those components that are inadequate while retaining the basic form of buildings’ lateral force resisting system.

Local improvements that can be considered include improving component connectivity, component strength, and/or component deformation capacity [20].

This strategy can be a cost-effective method to improve the seismic performance of a building when only a limited number of components are inadequate. Local strengthening could include measures such as adding a plywood overlay diaphragm over an existing timber floor or by adding concrete facings to the column elements of heavily perforated wall-type of structures.

Local improvements that improve the deformation capacity or ductility of components can allow them to survive large displacements without necessarily increasing the component strength. For example, placing steel jackets around reinforced concrete columns can allow the columns to deform without loss of strength through spalling, degrading flexural reinforcement splices or shear failure in plastic hinge zones. Local intervention methods are aimed at increasing the deformation capacity of deficient components so that they will not reach their specified limit state as the building responds at the design level. Common approaches mainly include [12]:

- Steel jacketing and
- Externally bonded Fiber Reinforced Polymer, FRP

6.3 Global Structural Strengthening and Stiffening

Some flexible structures will have poor seismic performance because critical components or elements do not have adequate ductility to resist the large seismic deformations usually associated with that type of structure.

For structures with many such elements, a cost-effective way to improve performance is to stiffen the structure so as to reduce the ductility demand on those critical components. By stiffening the structure, the building period will reduce and the elastic strength demands on the lateral force resisting system will typically increase. Stiffening a structure is usually accompanied with an increase in seismic strength.

Construction of new braced frames, moment resulting frames or shear walls within an existing structure are effective methods for adding both additional stiffness and strength.

By providing supplementary strength to the lateral force resisting systems, it is possible to raise the threshold of seismic intensity at which the onset of damage occurs.
Care is needed to ensure that the new strengthening elements are compatible with the stiffness of the existing elements so as to avoid premature or brittle failure of those elements [20].

Global intervention methods involve a global modification of the structural system; such modification is designed so that the design demands, often denoted by target displacement, on the existing structural and non-structural components are less than structural capacities.

Common approaches mainly include [12]:

- Reinforced Concrete (RC) jacketing,
- Addition of walls, steel bracing and
- Base isolation

6.3.1 Seismic Isolation

An alternative to strengthening a weak building is to substantially isolate it from damaging seismic ground motions. Base isolation produces a structural system, incorporating superstructure and isolation bearings, with a fundament response that corresponds to nearly rigid body translation of the superstructure above the isolation plane.

By base isolating a building the building period is usually extended out to 2 or 3 seconds – substantially reducing the seismic response into the super structure. Also, the isolation bearings are usually designed and built to incorporate a high level of seismic damping that further reduces the seismic response into the superstructure. The seismic demands on the superstructure, the non-structural components and contents are greatly reduced [20].

Base isolation is often an appropriate strategy to achieve the enhanced levels of seismic protection required for heritage buildings, buildings housing valuable collections or critical equipment. Most of the seismic deformation induced in a base isolated system occurs over the height of the bearings. Deformations of up to 300–600 mm are common with base isolated buildings.

Base isolation is most effective for relatively stiff low height buildings with a large seismic mass. This technique is less effective for light, flexible structures and tall buildings. Base isolation of existing buildings is technically complex, and it usually involves very detailed and careful supporting and foundation strengthening techniques. It can be a relatively costly technique to implement [20].

6.4 Reduction of Seismic Action Effects Through Retrofitting

In this strategy, seismic deformation demands on existing structural or (drift sensitive) non-structural elements are reduced below the corresponding capacities, which may remain unchanged. Absolute displacements are also reduced, decreasing the likelihood of pounding with adjacent buildings. Shear force demands cannot decrease, unless the member in question is kept in the elastic region.

The most effective and common means for the reduction of seismic deformation demands is by increasing the global lateral stiffness. Normally this brings about an increase in global lateral strength, which, however, should be seen as a by-product and not as the main target of the retrofitting.

In order of decreasing effectiveness, global lateral stiffness may be increased by [11]:

1. Adding a whole new lateral-load-resisting system to take almost the full seismic action. This system may consist of:
    - steel bracing,
    - new concrete walls
    - new moment frames, or combinations thereof.

The new elements are normally placed at the perimeter, to facilitate their foundation and to limit disruption of use of the building (under certain conditions, operation may continue during retrofitting). The new system can overshadow completely any irregularities in plan or elevation.

Critical elements in this approach are the foundation of the new lateral-load-resisting system and the connection to the existing system for the transfer of inertia forces. This approach lends itself to application of force-based retrofitting. In that case the new system is designed for ductility in full accordance with a code for new buildings, while the existing elements are considered as "secondary" and verified as such.

2. Adding new elements
   - new concrete walls,
   - or steel bracing,

to supplement the existing structural system. The new elements may be used to advantage to balance a strongly asymmetric layout in plan, or to eliminate soft/weak story. If the contribution of the added elements to lateral stiffness is large, this approach may be considered as a scaled-down version of approach 1.

3. Converting non-structural infill walls into structural elements, integrating them with the surrounding frame.

The approach has many aspects in common with 3 above. Note that, if the overlays added to the infills contain curtains of light reinforcement, detailing aspects (e.g., the connection or anchorage of the reinforcement to the frame and how it affects the behavior and the verifications; corrosion protection of the reinforcement, in view of the small thickness of the overlay, etc.) become very important.

4. Concrete jacketing, mainly of columns.

This is closer to retrofitting for the purposes of increasing the capacities. Only when practically all columns are jacketed (e.g., when architectural reasons do not allow adding new walls or steel bracing, or when there is wide-spread reinforcement corrosion), it might also be considered as part of a strategy to reduce seismic demands, albeit not the most cost-effective and less disruptive one.

6.5 Upgrading of Member Capacities

The deformation capacity and shear strength of individual members may be significantly upgraded by Fiber Reinforced Polymer or FRP-wrapping, without modifying at all their stiffness. Concrete jackets also improve deformation capacity and shear strength, but increase stiffness as well. So, when applied to many elements they also reduce deformation demands, not only locally but also globally. Improvement of certain details (e.g., of poor connections between the floor diaphragms and the lateral-load-resisting system or within diaphragms, see Section 6.7.4) may also be considered to belong to this retrofitting strategy [11].

Modification of existing components uses up less floor area and does not require closing openings. So, it is, in general, more convenient for the future functionality of the building than the addition of new elements, or of a large concrete volume to existing members to increase stiffness and reduce seismic demands.

Retrofitting via enhancement of member capacities alone makes more sense when deficiencies are limited to few members or connections or to part of the structure (a story or one side in plan). It may also be adopted when it is not feasible to add new elements (e.g., because of architectural constraints) and/or provide a proper foundation for them [11].

Unless very specific and substantial deficiencies are identified in some beams, upgrading of existing members may be limited to vertical elements, possibly including their joints with the beams. Due to the integral connection of the beams with the slabs, upgrading a beam is technically more difficult than upgrading a column or a wall. Besides, experience from past earthquakes shows that damage in beams is far less common than in columns and its impact on global stability minor.

Moreover, the design of beams for gravity loads normally provides sufficient top reinforcement at the supports (supplemented by the slab bars within a sizable effective flange width) and substantial shear reinforcement in the form of stirrups which are closed at the critical side (the bottom one). What is missing in such beams is continuity and anchorage of bottom bars over the supports.

However, bar pull-out under sagging moments, if it occurs, only increases the lateral deformability of frames. Another weak point is the poor deformation capacity of the bottom flange in compression, in plastic hinges under hogging moments, which however, not always take place.

Note that concrete jacketing of the columns into which a beam frames, improves, even though indirectly, anchorage of its bottom bars and confinement of its bottom flange. Last but not least, the main hazard for existing buildings is posed by too much, rather than by too little, moment resistance of beams with respect to columns [11].

Existing components may sometimes be modified to improve not their own performance but that of elements they connect to. For example, a weak-beam/strong column combination may be achieved by cutting beam longitudinal bars at the support by the column, provided that this is acceptable for gravity load resistance.

6.6 Strengthening Building Elements

Techniques for strengthening building elements are included in Table 4, covering the following:

- columns/piers
- beams
- beam/column joints
- floor roof and ceiling diaphragms
- shear walls.

Table 4: Summary of Techniques for strengthening building elements

Description	Design comment
1 **Columns/piers** 1.1 **Concrete columns steel jackets** Seismic behavior of columns can be improved using circular or elliptically shaped thin steel jackets to encase existing rectangular or circular columns. The jackets are continuously site welded then grout or concrete filled. The jackets will normally extend from floor level to the undersides of the beams above.	Steel jackets can be designed to increase column confinement, provide restraint against buckling of longitudinal bars, provide additional shear strength and provide additional lap bond strength. Generally, the steel jackets don't increase the flexural strength of the column and but they will usually reduce the effective plastic hinge length of the column in column yielding sideways mechanisms.
1.2 **Concrete columns – composite fiber wrapping** Seismic behavior of circular and/or rectangular columns can be improved by wrapping the columns with synthetic fibers bedded into an epoxy or other bonding material. High strength composite fibers such as carbon fiber or glass fiber are generally used. The fiber wraps are unidirectional to provide good confinement. Rectangular column corners need to be radiused to suit the bend radii of the specified fibers.	The fiber wraps can be designed to increase column confinement, provide restraint against buckling of longitudinal bars and provide additional shear strength and lap bond strength. Generally, the fiber wraps don't increase the flexural strength nor stiffness of the columns and they will usually allow the columns to develop full length plastic hinge zones in column yielding sideways mechanisms. Refer to the manufacturers of the fibers for specific design guidance and construction specification requirements.
1.3 Additional Concrete Jackets/Skins to **Columns and Piers** Concrete jacketing can be used to improve the deformation and shear capacity of columns and piers. The concrete jacketing incorporates transverse confinement reinforcement at reasonably close centers, comprising outer hoops around the perimeter of the existing column section and, commonly, cross ties drilled and anchored into the core of the existing column or passed right through the column.	The concrete jacketing is designed to increase column confinement, provide restraint against buckling of longitudinal bars in the existing column section, provide additional shear strength and lap bond strength. Additional flexural strength can be provided where the longitudinal reinforcement (and jacketing) is continuous through the floor system at each level. If the purpose of the jacketing is to increase the ductility but not the flexural strength of the column, the longitudinal reinforcement in the concrete jackets should be discontinued a short distance from the connection with adjacent components

Nominal longitudinal reinforcement is provided to support the transverse stirruping but can be used to improve flexural strength where fully anchored into adequate foundations and/or continuous through the floor system at each level.	
	Concrete jackets placed to improve ductility may also enhance the flexural strength due to the increase in section size. Where jacketing is not continuous this may shift the ductility demands to adjacent sections. This needs to be checked and appropriate steps such as extending the extent of jacketing considered. Measures need to be implemented to provide shear transfer between new and existing materials where composite action is required, such as for increase in flexural strength, such as scrabbling of the surface of the existing column.
2.2 Concrete beams **Composite fiber wrapping for** **shear/confinement** Composite fiber hoops are tightly wrapped completely around the concrete beams at regular centers to act as stirrups for shear enhancement and to provide anti buckling restraint and confinement to the longitudinal beam bars. Holes are drilled at regular centers through the floor slab adjacent to the sides of the beam. The top and bottom edges of the beam are carefully radiused to suit the fiber wrapping.	It is usually necessary to fiber wrap right around the concrete beams to develop strut-tie actions to improve shear strength and to provide the anti-buckling restraint over the length of the plastic hinge zones. The spacing of the hoops of fiber is determined by shear and anti-buckling requirements. Refer to the manufacturers of the fibers for specific design guidance and construction specification requirements

3 Beam/column joints **3.1 Steel or concrete jacketing of joint zones** Where concrete jacketing is used to increase the flexural strength of columns, jacketing needs to be passed through the floor system to ensure the transfer of enhanced strength between columns and beams in addition to improving confinement. The new column reinforcement is passed through the floor system and encased in concrete jacketing. Steel jacketing may be used to increase the flexural strength of columns at each level using longitudinal flat steel plates or angle sections at column corners passed through the floor system and transversely linked by steel straps or rods passed through holes drilled though the adjacent beams.	Where used for enhancement of flexural strength with jacketing, longitudinal reinforcement or steel strap reinforcement is passed and grouted through holes cored or broken through the floor slab adjacent the corners of the existing column section. The transverse reinforcement to jacketing is commonly installed though holes drilled through the concrete beams framing into the column at each floor level.
4. Diaphragms **4.1 Concrete Diaphragm struts and ties** Diaphragms transmit inertial forces in a structure to the lateral load resisting systems. Concrete diaphragms typically comprise slabs, collectors and chords The strength of diaphragms may be enhanced by the provision of additional reinforcement and concrete encasement or of structural steel plate or alternative sections along appropriate strut and tie lines to the slab diaphragm. Shear transfer to/from the existing diaphragm slab can be by reinforcing rods epoxy grouted into the existing concrete slab and beam members or by epoxy fixing and bolting steel plate sections to the slab.	A strut and tie approach to diaphragm design is becoming more commonly adopted in building design, particularly in slabs with major openings, irregular floor plans and irregular spaced lateral load resisting systems. Improvement of the strength of the individual strut and tie components of a diaphragm will often prove more cost effective than alternatives. Diaphragm thickness may be increased as an alternative approach but the added weight will increase the seismic load as well as increase footing loadings.

5 Shear walls **5.1 Concrete Skin Walls** Concrete "skin" walls are often used to increase the shear and flexural strength of existing walls or heavily perforated wall type structures. Constructing a "skin" on the inside (or outside) of these walls with carefully designed shear reinforcement can significantly increase the shear capacity and ductility of these walls. Concrete skin walls are also used to enhance the shear strength (and sometimes flexural strength) of plain cantilever walls.	Many existing concrete buildings have heavily perforated walls for regular patterns of windows or doors. The concrete piers between the openings are often relatively thin and prone to diagonal shear cracking as inelastic seismic actions are concentrated in the piers. Often, relatively little flexural reinforcement is required (or desired) to limit the over strength shear capacity of the piers. The flexural reinforcing in the existing walls is often adequate and the focus of the strengthening regime is to provide over strength shear reinforcement and detailing for ductility.
5.2 Composite fiber overlays The use of high strength composite overlays, such as carbon or glass fiber sheets or bands, epoxied to the surface can be used to enhance the stiffness and strength of existing concrete and masonry walls. They are used as tensioning reinforcing and can therefore increase both the in plane and out of plane strength of walls.	Carbon or glass fibers, woven into fabric sheets are applied to the surface of the wall using an epoxy resin binder and can be orientated in one or two directions. Several layers and orientations can be used depending on the design requirements. Carbon fibers have a modulus of elasticity and tensile strength greater than that of steel, whilst glass fibers have a lower modulus of elasticity and tensile strength. Both glass and carbon fibers exhibit brittle behavior in tension. Debonding of the fibers from the wall usually results under out of plane loadings.

6.7 Concrete Jacketing

Jacketing existing beams, columns, or joints can be done with new steel or reinforced concrete overlays. Jacketing may serve to increase flexural strength and ductility, and shear strength; to improve longitudinal reinforcement development or splicing; and to combine these effects. Although jacketing can be a technically effective procedure, when several components must be jacketed, it may not be cost effective, and it can also be very disruptive to building occupants [19]. Where jackets are used to increase flexural strength, and in some other cases requiring composite action, appropriate measures should be implemented to provide shear transfer between new and existing materials.

These measures may include:

- For concrete jackets, roughening the surface of the existing concrete prior to concrete placement, and using dowels to improve shear transfer strength when the jacket does not surround the component
- For steel jackets, using epoxy to effectively bond the steel to the concrete, and non-shrink grout or dry pack plus bolts or other anchorage devices

Where the objective is to increase component flexural strength, the technique must provide continuity across beam-column connections so that the enhanced strength can be transferred to adjacent framing components (Alcocer and Jirsa, 1993; Corazao and Durrani, 1989; Rodriguez and Park, 1992; Krause and Wight, 1990; Stoppenhagen and Jirsa, 1987). For columns, approaches include the following:

- New longitudinal reinforcement can be passed through the floor system and encased in a reinforced concrete jacket.
- Steel sections flanking the existing column can be connected to it to ensure composite action, and pass through the floor system to provide continuity. Similar approaches may be used for beams, including the addition of straps or continuous reinforcement across joints where beam bottom reinforcement is discontinuous.

Where the objective is to increase flexural ductility, either reinforced concrete or steel jackets can be added to deficient sections (Aboutaha et al., 1994). If the jacket completely surrounds the component or, in the case of beams, the jacket surrounds three faces and is anchored into the slab, only a nominal connection is required between existing and new materials. Concrete jackets should be reinforced with transverse reinforcement and nominal longitudinal reinforcement. Steel jackets may comprise bands or full-height jackets made of steel plates or shells; anchorage may be necessary along the side face of flat steel plates to improve confining action, and stiffeners may be required for thin plates. The space between steel jackets and existing concrete should be filled with non-shrink grout [19].

If the purpose of the jacket is to increase the flexural ductility but not increase the flexural strength, the longitudinal reinforcement in concrete jackets and steel in steel jackets should be discontinued a short distance (about 50 mm) from the connection with adjacent components. Concrete jackets placed to improve ductility may also enhance flexural strength, which may shift the ductility demands to adjacent sections, and this aspect should be checked and appropriate actions taken. In general, a jacket should extend from critical sections a distance equal to at least 1.5 times the cross-sectional dimension measured in the direction of the lateral load [19].

Concrete jackets are applied to columns and walls for all or some of the following purposes:

- increasing the bearing capacity,
- increasing the flexural and/or shear strength,
- increasing the deformation capacity,
- improving the strength of deficient lap splices

The thickness of the jackets should be such as to allow for placement of both longitudinal and transverse reinforcement with an adequate cover. When jackets aim at increasing flexural strength, longitudinal bars should be continued to the adjacent story through holes piercing the slab, while horizontal ties should be placed in the joint region through horizontal holes drilled in the beams. Ties can be omitted in the case of fully confined interior joints [3].

6.7.1 Enhancement of strength and deformation capacities

For the purpose of evaluating strength and deformation capacities of jacketed elements, the following approximate simplifying assumptions may be made [2]:

- the jacketed element behaves monolithically, with full composite action between old and new concrete;
- the fact that axial load is originally applied to the old column alone is disregarded, and the full axial load is assumed to act on the jacketed element;
- the concrete properties of the jacket are considered to apply over the full section of the element.

6.7.2 Advantages and Disadvantages of Concrete Jackets

Owing to their cost-effectiveness, concrete jackets are still the method of choice for seismic upgrading of individual concrete members. There are several reasons [11]:

1. Retrofitting, and especially modification of existing members, does not lend itself to prefabrication in shop. So, concrete is the prime candidate, as it is the most common structural material for field fabrication and application.
2. Concrete jacketing is suitable technique for retrofitting severely damaged members, as crushed and removed concrete is replaced while casting or concreting the jacket, while buckled or fractured bars do not need to be fully restored if replaced by the new reinforcement of the jacket.
3. Structural concrete is versatile and can adapt to almost any shape, e.g., in order to fully encapsulate existing members and joints and provide structural continuity between different components (between a joint and the adjoining members, between members in adjacent story, etc.).

A concrete jacket can, through the appropriate reinforcement, have multiple effects. It is the only means to improve at the same time:

- stiffness,
- shear strength,
- deformation capacity,
- anchorage or continuity of reinforcement in anchorage or splicing zones,
- moment resistance (even turning a weak-column/strong-beam frame into a strong-column/weak-beam one),
- shear strength and bond in joints through which the jacket continues, and
- protection of the old reinforcement from (further) corrosion.

4. Stiffness and flexural resistance are enhanced by the increased cross-section and the added longitudinal reinforcement, which, very importantly and unlike for other retrofit techniques of individual components, can easily extend beyond the member end into and through the joint.

5. The main contribution to shear strength, deformation capacity and anchorage or splicing of reinforcement comes from the added transverse reinforcement, which works in shear, against buckling and for confinement. The added concrete is also a factor there. The increased dimensions of a joint when a jacket continues into and past it, provide more length for bond along old bars going through the joint and improve the joint shear strength. They also make room for adding transverse reinforcement in the joint.
6. Finally, if the jacket concrete is of sufficiently low porosity, it can prevent or arrest corrosion of old reinforcement even in carbonated concrete. As a minimum, it reduces markedly the mechanical and aesthetic consequences of any corrosion that may go on.

Concrete jacketing may be considered to serve at the same time both types of retrofitting strategies mentioned above. By increasing the stiffness, it reduces seismic displacement and deformation demands. Besides, it is very effective in enhancing the force and deformation capacity of the jacketed member.

From the technical point of view, the multiple effectiveness of concrete jackets is what mainly distinguishes them from the other seismic retrofitting techniques for individual concrete members, which cannot readily extend beyond the member end and into a joint region. Other techniques mainly enhance some or all of the properties above, but normally not the flexural strength, the resistance of the joints themselves or the corrosion protection all along the member.

Reinforced Concrete jackets have certain handicaps, compared to other member modification techniques:

- They considerably increase member cross sectional dimensions. This may be a serious drawback when space, especially floor area, is at a premium.
- They normally cause the largest inconvenience and the lengthiest disruption of occupancy, produce the largest amount of dust and debris (especially if shotcrete is used) and cause the most noise pollution and safety or health hazards for the workers.

6.8 Technological and Construction Aspects

The concrete overlay of the jacket should be at least 75–100 mm, to provide sufficient cover of the new reinforcement and space for 135◦-hooks at tie ends (See Figure below) [11]. For this range of thickness shotcrete is more convenient. Thicker overlays are normally cast-in-place.

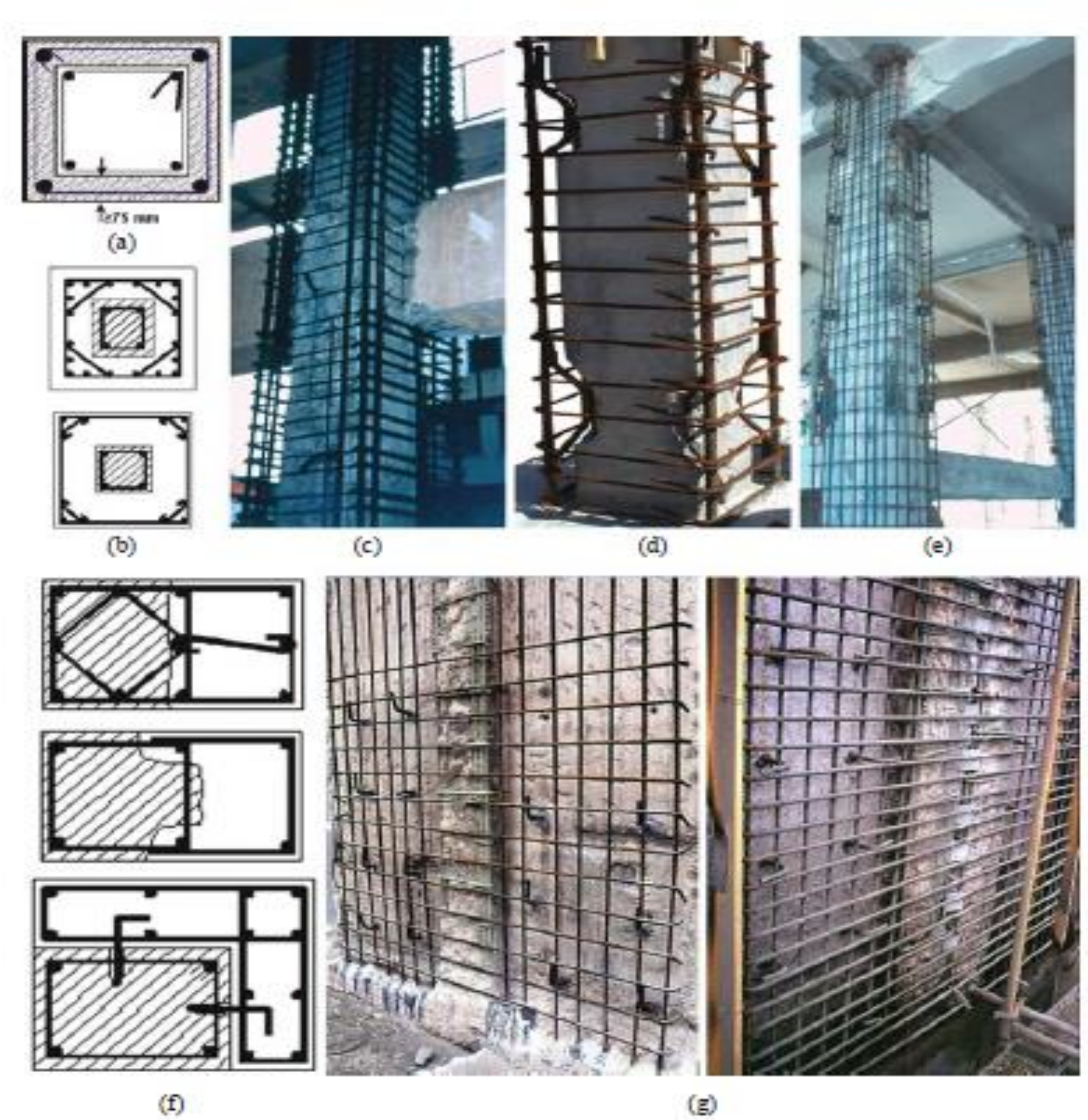


Table 5: Concrete jackets in columns: (a) simplest case; (b) jacket bars bundled near corners, engaged by cross-ties or octagonal tie; (c) jacket bars bundled at corners, dowels at interface with old column; (d) U-bars welded to corner bars; (e) steel plates welded to corner bars; (f) one-or two- sided jackets; (g) one-sided concrete overlay with single curtain of two-way reinforcement at exterior face of perimeter walls.

For the moment resistance of vertical elements to increase, longitudinal reinforcement should be continued to the adjacent story through holes or slots in the slab. To avoid perforating the beams on all sides of the cross-section into which a beam frames, jacket bars continuing through the slab should be concentrated near the corners of the new section, often in bundles (Figure (b) and (c)). Jacket vertical bars may be anchored into a foundation element either:

- by enlarging the foundation element to accommodate anchorage of the jacket bars in new concrete there (possibly increasing at the same time the capacity of the foundation element to meet the larger moment demands from the jacketed vertical member), or
- by fastening (e.g. through epoxy) starter bars within vertical holes drilled in the foundation element, to be lap-spliced with the jacket vertical bars outside the plastic hinge that may form at the bottom of the retrofitted element.

A closed perimeter tie around the vertical bars of a column jacket restrains them against buckling, adds shear strength and confines the concrete. If we need multiple ties but do not want to drill holes and thread cross-links through the core of the old column, we can supplement the perimeter tie with an octagonal hoop outside the old column, which restrains buckling of any vertical bars close to, but not exactly at the corner.

Instead of the octagonal tie, short corner ties at 45◦ to the perimeter may be used, engaging in 90–135◦ hooks the two bars adjacent to the corner bar (Figure. (b)). A diamond-shaped tie can be used only when the side of the jacketed column is at least twice as long as that of the old column. However, it is meaningless unless the jacket has mid-side bars to be restrained.

Three-sided jackets are sufficient for beams integral with the slab. However, one-, two-, or three-sided jackets not fully surrounding an old column are much less effective than full jackets. Besides, their seismic behavior is little known. If a full jacket around a column is not feasible, the old reinforcement should be exposed and the new ties welded to the old ones or bent around the old vertical bars (Figure(f)).

One-sided jackets are easy to add to the exterior face of perimeter members in a building. If the member is a wall, a one-sided jacket having full horizontal and vertical reinforcement and well connected to the old wall through dowels may play the role of an appropriately reinforced new wall (Figure(g)). The main contributions of the old wall are its concrete and the connection it provides to the rest of the structural system and to the foundation for the transfer of seismic forces.

Past guidance documents for concrete jackets – and past practice alike – include measures for shear connection of the old and the new concrete. Connecting the (corner) bars of the jacket to the (exposed for this purpose) longitudinal bars of the old column, by lap-welding both to Z- or U-shaped steel inserts is commonly recommended and applied (see Figure. (d) and (e)).

Alternatively, the surface of the old element may be roughened and/or dowels may be driven into it (Figure (c) and (g)). The dowels are epoxy-grouted in holes drilled into the old element and protrude in the overlay of new concrete for almost its full thickness. As this thickness is usually small, the dowel is often bent at 90◦ for anchorage in the new concrete.

6.9 Steel jacketing

Steel jackets are applied to columns with the purpose of:

- increasing shear strength,
- improve the strength of deficient lap-splices, and
- increase ductility through confinement.

Steel jackets around rectangular columns are usually made up of four corner angles to which either continuous steel plates, or thicker discrete horizontal steel straps, are welded.

Corner angles may be epoxy-bonded to the concrete, or just made to adhere to it without gaps along the entire height. Straps may be pre-heated just prior to welding, in order to provide afterwards some positive confinement on the column.

6.9.1 Shear strength

The contribution of the jacket to shear strength may be considered as additive to existing strength, provided the jacket remains well within the elastic range. This condition is necessary for the jacket to be able to control the width of internal cracks and to ensure the integrity of the concrete, thus allowing the original shear resisting mechanism to continue to operate.

6.9.2 Clamping of lap-splices

Steel jackets can provide effective clamping in the regions of lap-splices, so as to achieve high cyclic deformation capacity. For this result to be obtained it is necessary that:

- the length of the jacket exceeds by no less than 50% the length of the splice region,
- the jacket is pressured against the faces of the column by at least two rows of bolts on each side normal to the direction of loading,
- when splicing occurs at the base of the column, the rows of bolts should be located one at the top of the splice region and another at 1/3 of that region, starting from the base.

6.10 FRP plating and wrapping

Externally bonded FRP have been used extensively in retrofitting reinforced concrete structures, though mostly in non-seismic cases. The main uses of FRP in seismic strengthening of existing reinforced concrete elements are the following:

- Enhancement of the shear capacity of columns and walls, by applying externally bonded FRP with the fibers in the hoop direction;
- Enhancement of the available ductility at beam or column ends, through added confinement in the form of FRP jackets, with the fibers placed along the perimeter;
- Prevention of lap splicing, through increased lap confinement again with the fibers along the perimeter.

6.10.1 Shear strength

Shear capacity of brittle components can be enhanced in beams, columns or shear walls through the application of FRP sheets. These can be applied either by fully wrapping the element, or by bonding them to the sides and the soffit of the beam (U shaped sheet), or by bonding them to the sides only.

6.10.2 Confinement action

The enhancement of deformation capacity is achieved through concrete confinement by means of FRP jackets. These are applied around the element to be strengthened in the potential plastic hinge region.

7.0 Design of structural intervention

7.1 Redesign Procedure

The redesign process shall cover the following steps:

a) Conceptual design

- Selection of techniques and/or materials, as well as of the type and configuration of the intervention.
- Preliminary estimation of dimensions of additional structural parts
- Preliminary estimation of the modified stiffness of the repaired/strengthened elements.

b) Analysis

The methods of analysis of the structure as redesigned shall be those indicated in 4.4 of EN 1998-1, as appropriate, considering the new characteristics of the building. In case the redesign consists in the addition of new structural elements intended to resist the entire seismic action, the latter should be designed using the seismic action, the method of analysis, and the verification procedures as in EN 1998-1.

c) Verifications

Safety verifications shall be carried out in accordance with 4.5 of EN 1998-1. For existing components, material safety factors γ_m shall be the same as in EN 1998-1, according to the level of knowledge specified.

Safety verifications for Nonlinear methods of analysis (static or dynamic)

The demands on both “ductile” and “brittle” components shall be those obtained from the analysis performed according to 4.4.4 or 4.4.5. Information on the evaluation of the capacity for both ductile and brittle components and mechanisms can be found in the relevant material-related Annexes, taking into account of 2.2.1(4).

8.0 Seismic Retrofit with Limited Replacement/Modification of Frame Members

Analysis of the reinforced concrete moment resisting frame of the example building indicated that the lateral moment resisting frames do not have the strength and details required for ductile behavior. This was due to the design of the structure using the previous code or the quality of supervision of the construction at the time. The analysis result showed that all frames are deficient in resisting the seismic actions defined according to the revised building code. Another analysis was done with modified cross sections and with the same seismic actions that were used to analyze the actual structure. The table below shows the type, dimension and reinforcement that is designed and used for the actual construction of the building. The linear static analysis of the building with the actual reinforcement and cross section, material property and loading using a design ground acceleration of 0.1 exhibited that the cross sections were overstressed and longitudinal reinforcement below the minimum requirement.

Table 6 Actual structural data for frame members

Designation	Type	Story	Column Dimension [mm]	Rebar
Column	C-1	Ground	400 x 700	28 ϕ 24
	C-2	Ground	400 x 700	24 ϕ 24
	C-3	Ground	400 x 700	16 ϕ 24
	C-4	Ground	400 x 600	16 ϕ 24
	C-1	First	400 x 600	24 ϕ 24
	C-2	First	400 x 600	16 ϕ 24
	C-3	First	400 x 600	16 ϕ 24
	C-4	First	400 x 500	12 ϕ 20
	C-1	Second	400 x 600	12 ϕ 20
	C-2	Second	400 x 500	12 ϕ 20
	C-3	Second	400 x 500	12 ϕ 20
	C-4	Second	400 x 400	12 ϕ 20
	C-1	Third	400 x 500	20 ϕ 20
	C-2	Third	400 x 500	12 ϕ 20
	C-3	Third	400 x 500	12 ϕ 20
	C-4	Third	400 x 400	12 ϕ 20

Designation	**Type**	**Story**	**Column Dimension [mm]**	**Rebar**
Column	C-1	Fourth	400 x 400	16 ϕ 20
	C-2	Fourth	400 x 400	12 ϕ 20
	C-3	Fourth	400 x 400	16 ϕ 16
	C-4	Fourth	400 x 400	12 ϕ 16
	C-1	Fifth	400 x 500	16 ϕ 20
	C-2	Fifth	400 x 400	12 ϕ 20
	C-3	Fifth	400 x 400	12 ϕ 16
	C-4	Fifth	400 x 400	12 ϕ 16
	C-1	Sixth	400 x 400	16 ϕ 16
	C-2	Sixth	300 x 300	12 ϕ 16
	C-3	Sixth	400 x 400	12 ϕ 16
	C-4	Sixth	400 x 400	8 ϕ 16
	C-1	Seventh	300 x 300	16 ϕ 16
	C-2	Seventh	300 x 300	12 ϕ 16
	C-3	Seventh	300 x 300	8 ϕ 16
	C-4	Seventh	300 x 300	8 ϕ 16
	C-1	Eighth	300 x 300	12 ϕ 16
	C-2	Eighth	300 x 300	12 ϕ 14
	C-3	Eighth	300 x 300	8 ϕ 16
	C-4	Eighth	300 x 300	8 ϕ 16
	C-1	Ninth	300 x 300	12 ϕ 14
	C-2	Ninth	300 x 300	8 ϕ 14
	C-3	Ninth	300 x 300	8 ϕ 14
	C-4	Ninth	300 x 300	8 ϕ 14
	C-1	Tenth	250 x 250	8 ϕ 14
	C-2	Tenth	250 x 250	8 ϕ 14
	C-3	Tenth	250 x 250	8 ϕ 14
	C-4	Tenth	250 x 250	8 ϕ 14

Table 7: Cross section dimension after redesign using the new code for frame on Axis 1

Designation	Type	Story	Column Dimension [mm]	**Location**	
Column	C-3	Ground	800 x 800	Axis A1	
	C-1	Ground	900 x 900	Axis B1	
	C-1	Ground	900 x 900	Axis C1	
	C-3	Ground	750 x 800	Axis D1	
	C-4	Ground	750 x 800	Axis E1	
	C-3	First	800 x 800	Axis A1	
	C-1	First	800x 800	Axis B1	
	C-1	First	750 x 800	Axis C1	
	C-3	First	750 x 800	Axis D1	
	C-4	First	750 x 800	Axis E1	
	C-3	Second	700 x 800	Axis A1	
	C-1	Second	800 x 800	Axis B1	
	C-1	Second	750 x 800	Axis C1	
	C-3	Second	750 x 800	Axis D1	
	C-4	Second	750 x 800	Axis E1	
	C-3	Third	700 x 800	Axis A1	
	C-1	Third	800 x 800	Axis B1	
	C-1	Third	750 x 800	Axis C1	
	C-3	Third	750 x 800	Axis D1	
	C-4	Third	750 x 800	Axis E1	
	C-3	Fourth	400 x 600	Axis A1	
	C-1	Fourth	700 x 800	Axis B1	
	C-1	Fourth	750 x 800	Axis C1	
	C-3	Fourth	750 x 800	Axis D1	
	C-4	Fourth	600 x 800	Axis E1	

Designation	Type	Story	Column Dimension [mm]	**Location**	
	C-3	Fifth	400 x 600	Axis A1	
	C-1	Fifth	700 x 800	Axis B1	
	C-1	Fifth	750 x 800	Axis C1	
	C-3	Fifth	700 x 800	Axis D1	
	C-4	Fifth	600 x 800	Axis E1	
Column	C-3	Sixth	400 x 600	Axis A1	
	C-1	Sixth	400 x 600	Axis B1	
	C-1	Sixth	750 x 800	Axis C1	
	C-3	Sixth	400 x 600	Axis D1	
	C-4	Sixth	400 x 600	Axis E1	
	C-3	Seventh	400 x 600	Axis A1	
	C-1	Seventh	400 x 500	Axis B1	
	C-1	Seventh	400 x 500	Axis C1	
	C-3	Seventh	400 x 500	Axis D1	
	C-4	Seventh	400 x 500	Axis E1	
	C-3	Eighth	400 x 600	Axis A1	
	C-1	Eighth	400 x 500	Axis B1	
	C-1	Eighth	400 x 500	Axis C1	
	C-3	Eighth	400 x 500	Axis D1	
	C-4	Eighth	400 x 500	Axis E1	
	C-3	Ninth	400 x 600	Axis A1	
	C-1	Ninth	400 x 500	Axis B1	
	C-1	Ninth	400 x 500	Axis C1	
	C-3	Ninth	400 x 500	Axis D1	
	C-4	Ninth	400 x 500	Axis E1	

Therefore, a detailed procedure is given for the retrofitting of the building, by modification and/or limited replacement of members on a perimeter frame for the required strength and ductility as per the result of the analysis output.

Out of different feasible schemes of strengthening of the building frame, modification using concrete jacketing technique is will be used. The modified structure is then checked using nonlinear static analysis or pushover analysis to meet the appropriate provisions of the codes.

The reason behind this is that same amount of ductility and strength can be given without doing much salvage of existing frame. Thus, the scheme is advantageous over other in the sense of its less necessity of salvage, shoring and less construction cost. But feasibility of the scheme is highly dependent on the design and detailing of the existing frame.

Figure 14: Concrete Jacketing in practice

Figure 15: Use of steel dowels and roughening the surface of an original column to achieve connection at the interface

Figure 16: Bent connecting steel bars

Figure 17: Total Jacket

Results of the pushover analysis for the modified structure as per the seismic analysis performed

The methodology followed to evaluate the performance of the redesigned structure is as follows:

1. A linear seismic analysis based on which a primary structural design is done.
2. Insertion of hinges determined based on the design and
3. A pushover analysis is carried out followed by
4. Modification of the design wherever necessary based on the later analysis.

In pushover analysis, in the global sense, it is the base shear versus roof top displacement taken as displacement of a point on the roof, located in plan at the center of mass. At local level, it is the hinge states to be examined and decided on the need for redesign or a retrofit. Based on the pushover analysis, the hinge states and status for selected frames, together with the various out puts are shown for reference.

8.2 Conclusion

It can be seen from the results of the pushover analysis that the status of all the hinges are with in LS for the Performance Point, taken at step 9 and 6 for PushX and PushY respectively with in IO performance level. Therefore, retrofitting of the building at member level using concrete jacketing by using the analysis output would enable the building to have a better performance for the ground acceleration and seismic zone as per the revised national code.

This project concludes with a recommendation that a well-structured building stock assessment is required to identify the status of reinforced concrete buildings which were designed and constructed before the revision so that action can be taken either retrofitting or demolishing of the building as per the results of the assessment to save human life. The different assessment and retrofitting techniques described in the project can be adopted in the future organized assessment rehabilitation of existing reinforced concrete buildings located especially in highly seismic zones of the country.

Annex

Hinge Response - B10H10 (Auto M3)

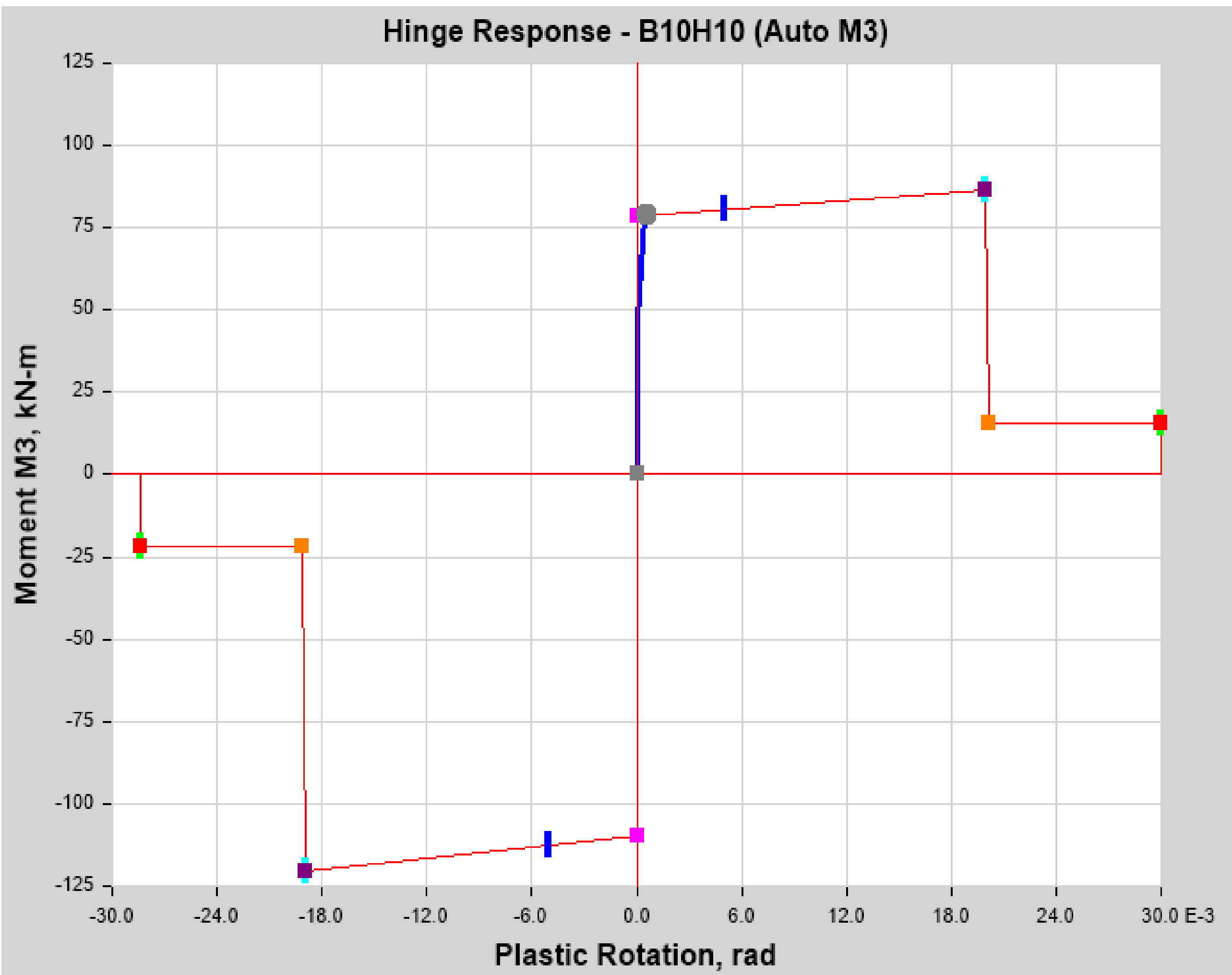
Summary Description

This is hinge response output for a specific hinge and a selected load case.

General Input Data

Load Case	PushY	Hinge	B10H10 (Auto M3)
Story	9THFLR	Hinge DOF	M3
Beam	B10	Hinge Rel. Dist.	0.95

Hinge Response Plot



Current Step Data

Step	6	Plastic Rotation Max	0.000554 rad
Moment M3	78.8517 kN-m	Plastic Rotation Min	0 rad
Plastic Rotation	0.000554 rad	Hinge State	B to <=C
		Hinge Status	A to <=IO

Response Values

Hinge Response Values

Step	M3 **kN**	R3 **Rad**	R3 Max **rad**	R3 Min **rad**	State	Status
0	0	0	0	0	A to <=B	A to <=IO
1	47.2555	0	0	0	A to <=B	A to <=IO
2	78.8119	0.000453	0.000453	0	B to <=C	A to <=IO
3	78.8317	0.000503	0.000503	0	B to <=C	A to <=IO
4	78.8339	0.000509	0.000509	0	B to <=C	A to <=IO
5	78.8363	0.000515	0.000515	0	B to <=C	A to <=IO
6	78.8517	0.000554	0.000554	0	B to <=C	A to <=IO

Hinge Response - B10H16 (Auto M3)

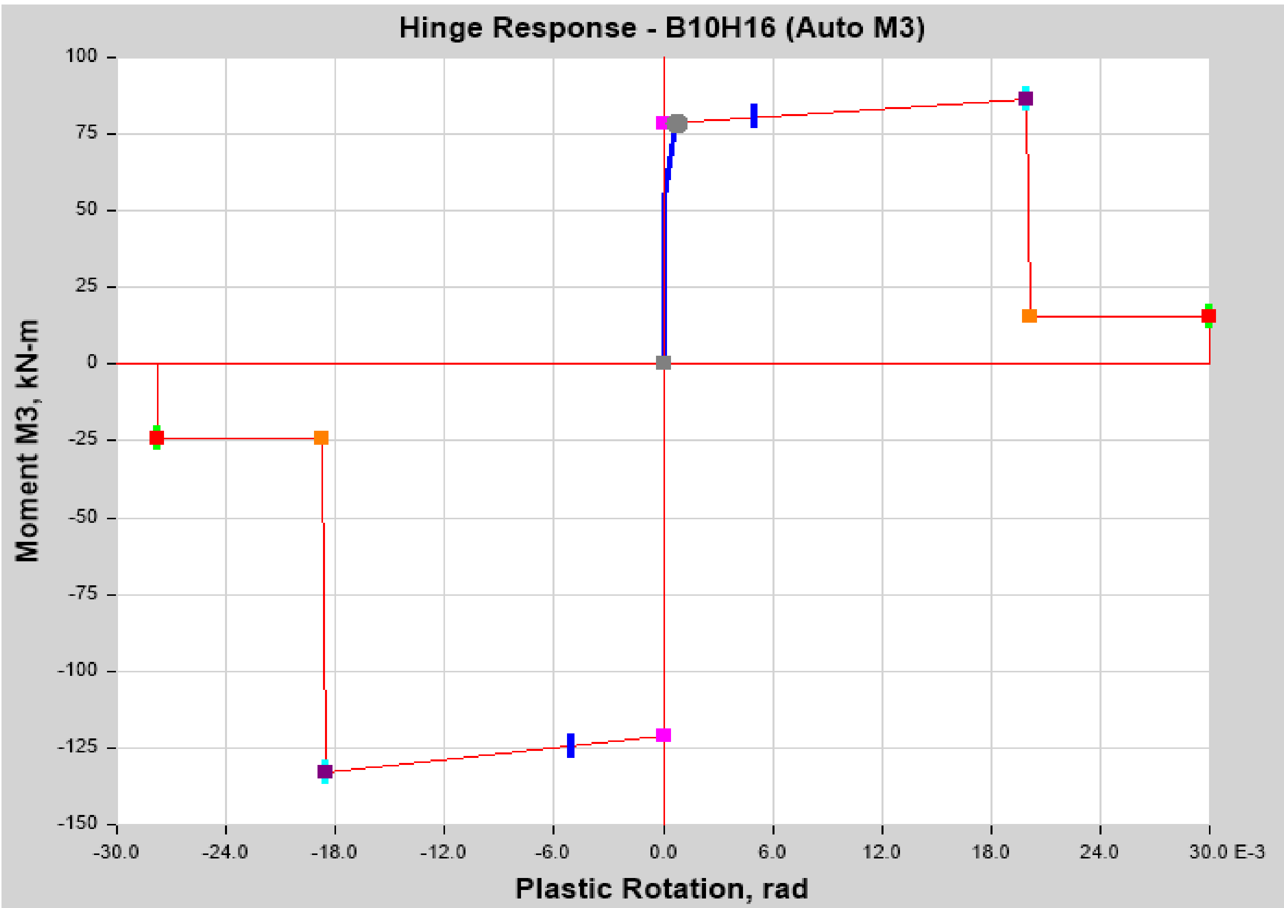
Summary Description

This is hinge response output for a specific hinge and a selected load case.

General Input Data

Load Case	PushY	Hinge	B10H16 (Auto M3)
Story	8THFLR	Hinge DOF	M3
Beam	B10	Hinge Rel. Dist.	0.95

Hinge Response Plot



Current Step Data

Step	6	Plastic Rotation Max	0.000758 rad
Moment M3	78.5473 kN-m	Plastic Rotation Min	0 rad
Plastic Rotation	0.000758 rad	Hinge State	B to <=C
		Hinge Status	A to <=IO

Response Values

Hinge Response Values

Step	**M3** **kN**	**R3** **rad**	**R3 Max** **rad**	**R3 Min** **rad**	**State**	**Status**
0	0	0	0	0	A to <=B	A to <=IO
1	54.2743	0	0	0	A to <=B	A to <=IO
2	78.5026	0.000644	0.000644	0	B to <=C	A to <=IO
3	78.5249	0.0007	0.0007	0	B to <=C	A to <=IO
4	78.5274	0.000707	0.000707	0	B to <=C	A to <=IO
5	78.53	0.000714	0.000714	0	B to <=C	A to <=IO
6	78.5473	0.000758	0.000758	0	B to <=C	A to <=IO

Hinge Response - B14H1 (Auto M3)

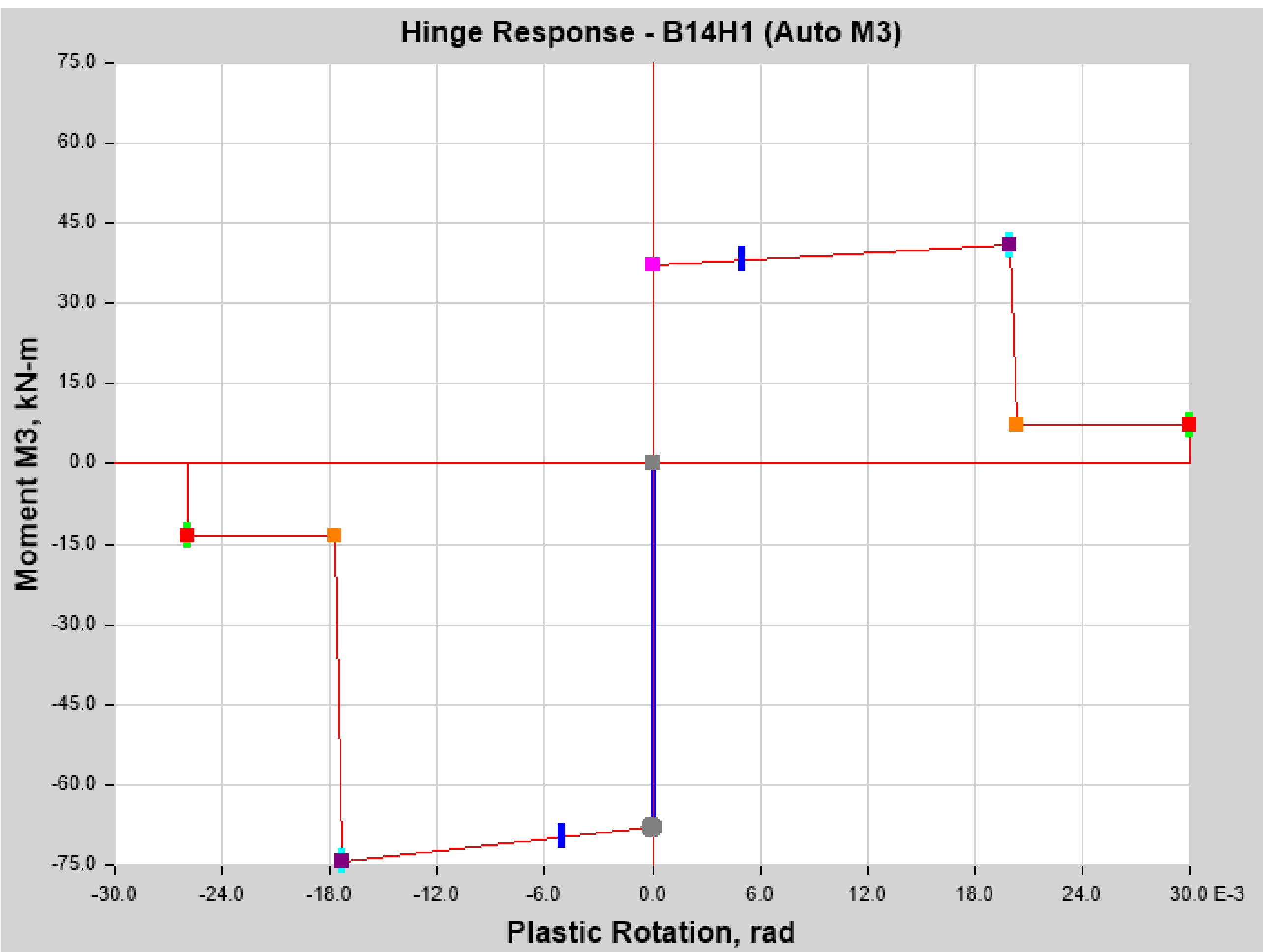
Summary Description

This is hinge response output for a specific hinge and a selected load case.

General Input Data

Load Case	PushY	Hinge	B14H1 (Auto M3)
Story	TTB	Hinge DOF	M3
Beam	B14	Hinge Rel. Dist.	0.05

Hinge Response Plot



Current Step Data

Step	6	Plastic Rotation Max	0 rad
Moment M3	-67.7247 kN-m	Plastic Rotation Min	-0.000024 rad
Plastic Rotation	-0.000024 rad	Hinge State	B to <=C
		Hinge Status	A to <=IO

Response Values

Hinge Response Values

Step	M3 kN	R3 rad	R3 Max rad	R3 Min rad	State	Status
0	0	0	0	0	A to <=B	A to <=IO
1	-24.7473	0	0	0	A to <=B	A to <=IO
2	-64.2927	0	0	0	A to <=B	A to <=IO
3	-66.5163	0	0	0	A to <=B	A to <=IO
4	-66.7565	0	0	0	A to <=B	A to <=IO
5	-67.0111	0	0	0	A to <=B	A to <=IO
6	-67.7247	-0.000024	0	-0.000024	B to <=C	A to <=IO

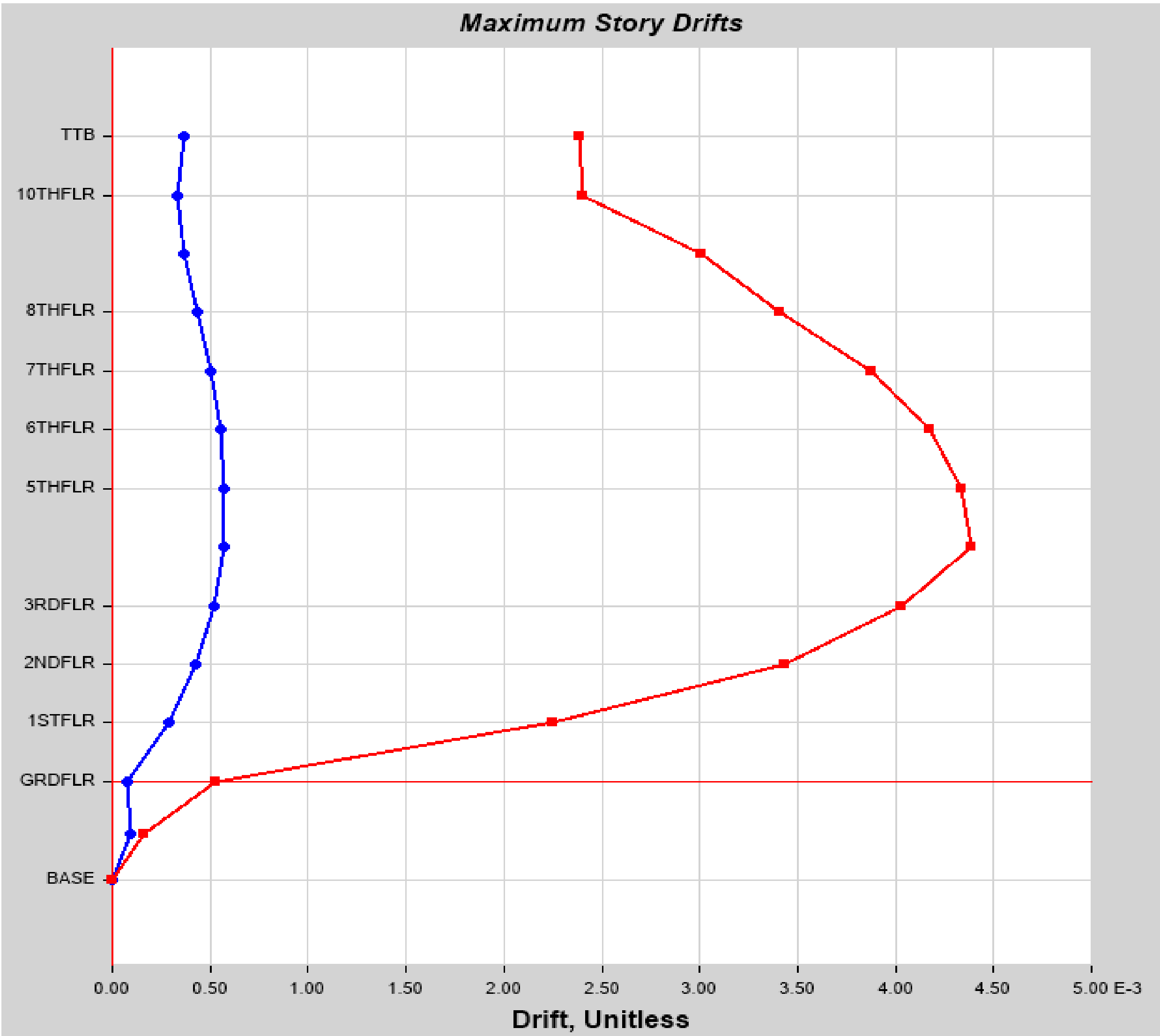
Story Response - Maximum Story Drifts

Summary Description

This is story response output for a specified range of stories and a selected load case or load combination.

Input Data

Name	StoryResp1		
Display Type	Max story drifts	Story Range	All Stories
Load Case	PushY	Top Story	TTB
Output Type	Step Number 6	Bottom Story	BASE



Story Response Values

Story	Elevation m	Location	X-Dir	Y-Dir
TTB	35.2	Top	0.000369	0.002385
10THFLR	32	Top	0.000329	0.002398
9THFLR	28.8	Top	0.000368	0.003005
8THFLR	25.6	Top	0.000434	0.003409
7THFLR	22.4	Top	0.000503	0.00388
6THFLR	19.2	Top	0.000551	0.004177
5THFLR	16	Top	0.000575	0.004332
4THFLR	12.8	Top	0.000569	0.004388
3RDFLR	9.6	Top	0.000517	0.004029
2NDFLR	6.4	Top	0.000426	0.003432
1STFLR	3.2	Top	0.000286	0.002249
GRDFLR	0	Top	0.00008	0.000531
GB	-2.87	Top	0.000093	0.000164
BASE	-5.37	Top	0	0

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
0	0	0	3080	0	0	0	0	3080	0	0	0	3080
1	-4.5	641.0475	3080	0	0	0	0	3080	0	0	0	3080
2	-9	1282.0949	3080	0	0	0	0	3080	0	0	0	3080
3	-13.5	1923.1424	3080	0	0	0	0	3080	0	0	0	3080
4	-18	2564.1898	3080	0	0	0	0	3080	0	0	0	3080
5	-22.5	3205.2373	3080	0	0	0	0	3080	0	0	0	3080
6	-25.53	3636.9417	3078	2	0	0	0	3080	0	0	0	3080

Table: Base shear Vs Monitored Displacement

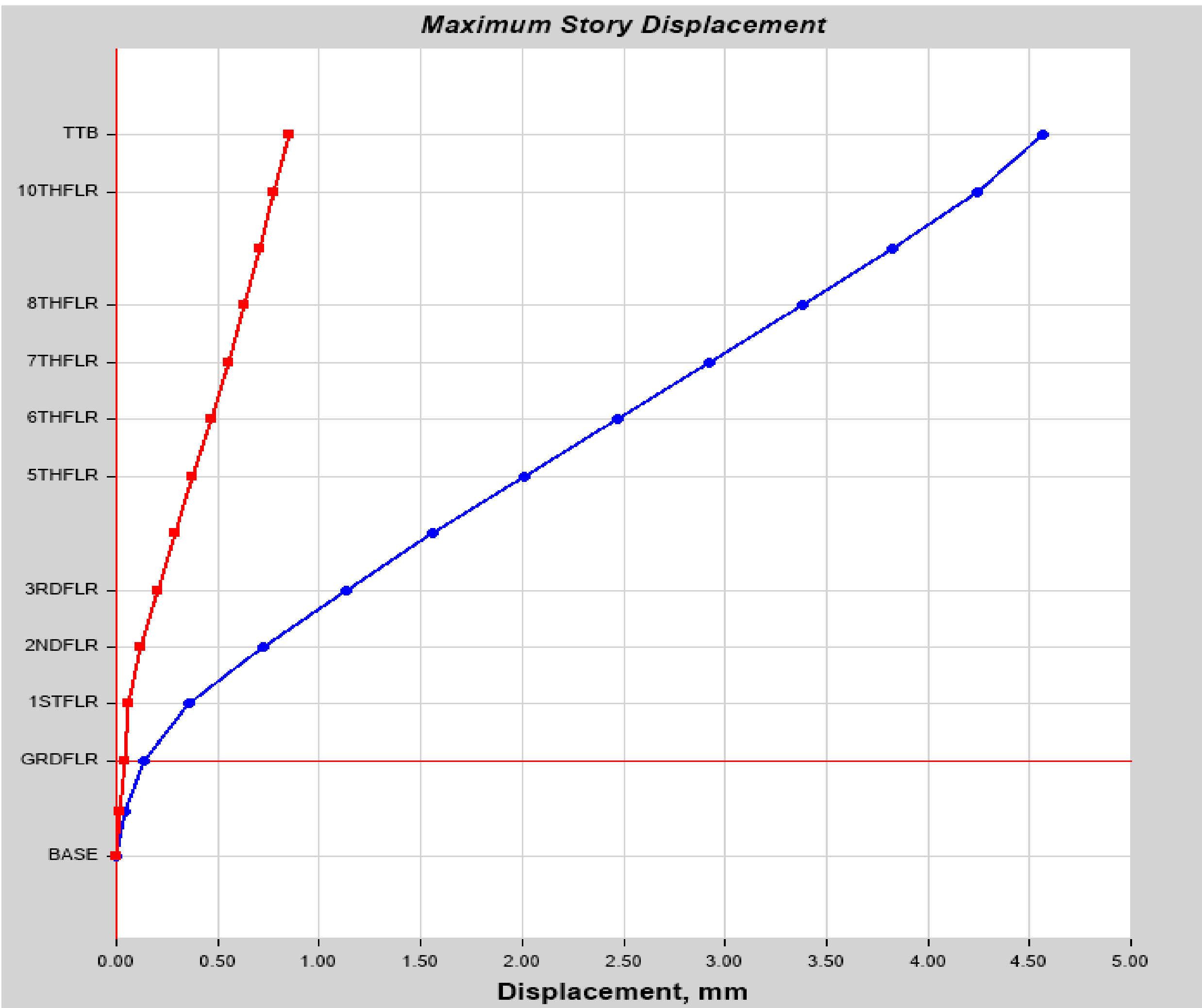
Story Response - Maximum Story Displacement

Summary Description

This is story response output for a specified range of stories and a selected load case or load combination.

Input Data

Name	StoryResp1		
Display Type	Max story displacement	Story Range	All Stories
Load Case	PushX	Top Story	TTB
Output Type	Step Number 1	Bottom Story	BASE



Tabulated Plot Coordinates

Story Response Values

Story	Elevation m	Location	X-Dir mm	Y-Dir mm
TTB	35.2	Top	4.568	0.85
10THFLR	32	Top	4.24	0.777
9THFLR	28.8	Top	3.824	0.705
8THFLR	25.6	Top	3.38	0.631
7THFLR	22.4	Top	2.92	0.551
6THFLR	19.2	Top	2.471	0.466
5THFLR	16	Top	2.012	0.376
4THFLR	12.8	Top	1.56	0.287
3RDFLR	9.6	Top	1.129	0.2
2NDFLR	6.4	Top	0.725	0.123
1STFLR	3.2	Top	0.354	0.064
GRDFLR	0	Top	0.14	0.04
GB	-2.87	Top	0.043	0.013
BASE	-5.37	Top	0	0

Pushover Curve - EC 8 2004 Target Displacement

Summary Description

This is the data for an EC 8 2004 target displacement pushover analysis.

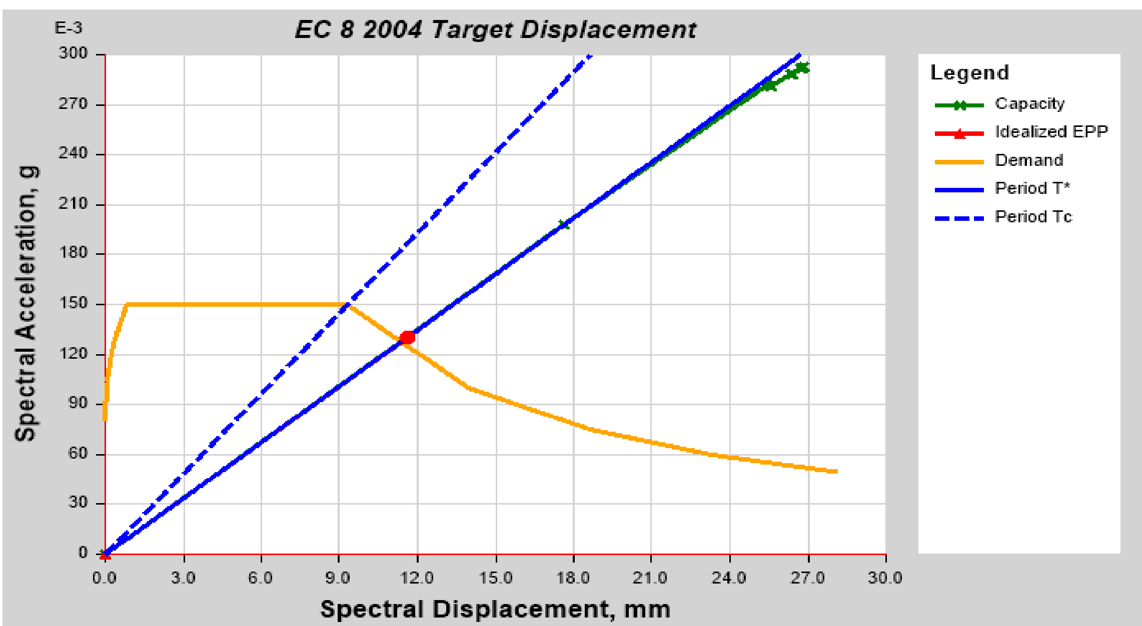
General Input Data

Name	Pushover		
Load Case	Push X	Plot Type	EC 8 2004 Target Displacement

Demand Spectrum Input Data

Source	EC 8 2004 Horizontal	Country	CEN Default
Spectrum Type	1	ag/g	0.1
Ground Type	B	q*	2

Pushover Plot



Target Displacement Results

Displacement, dt	**21.297 mm**	**Shear at dt**	**3293.0943 kN**

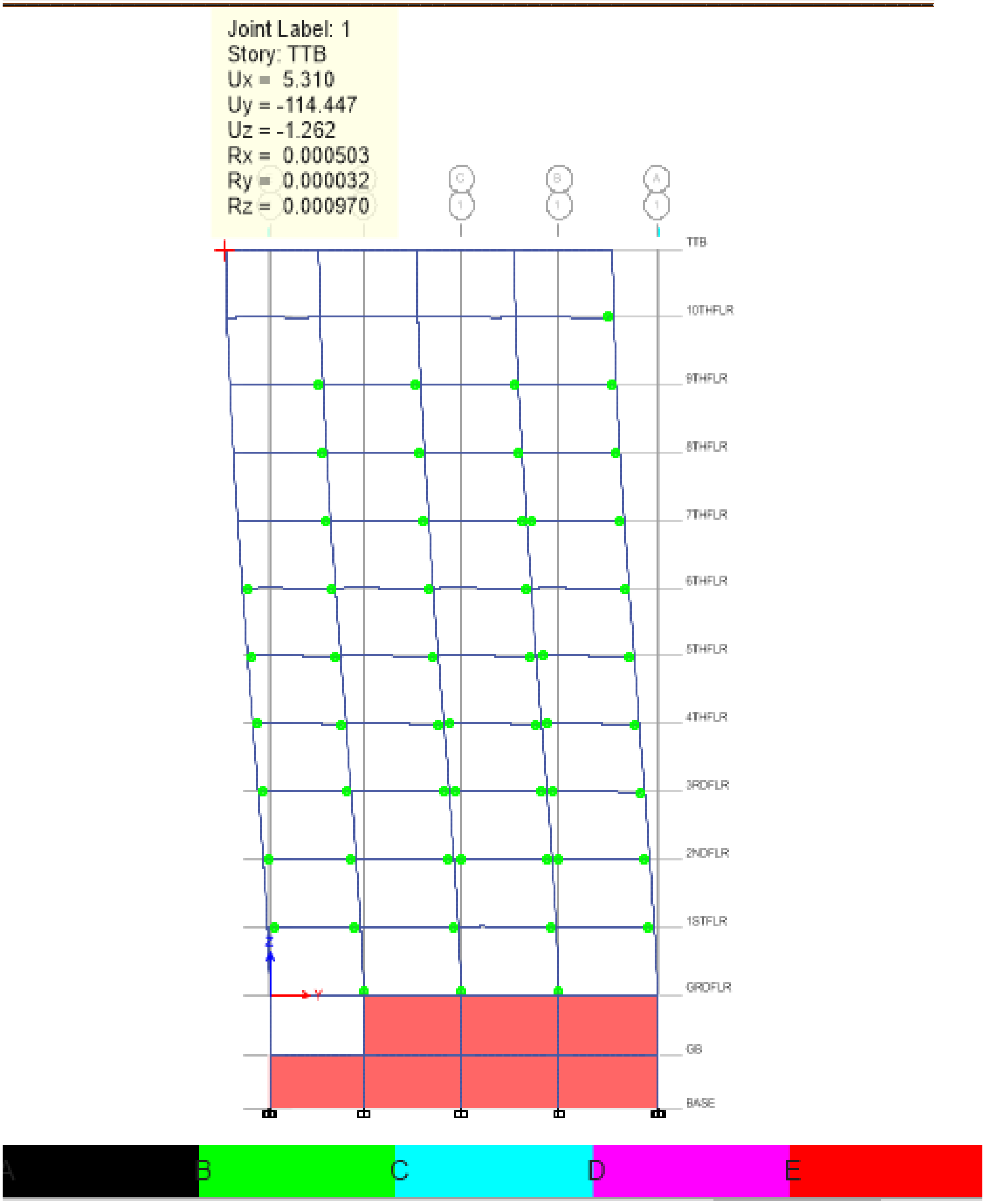
Calculated Parameters

Target Displacement Found	**Yes**	**Em***	**0.755 mm**
Fy*	0.13	T*	0.599 sec
dy*	11.599 mm	Se(T*)	0.13
det*	11.599 mm	Tc	0.5 sec
dt*	11.599 mm	Gamma	1.836028

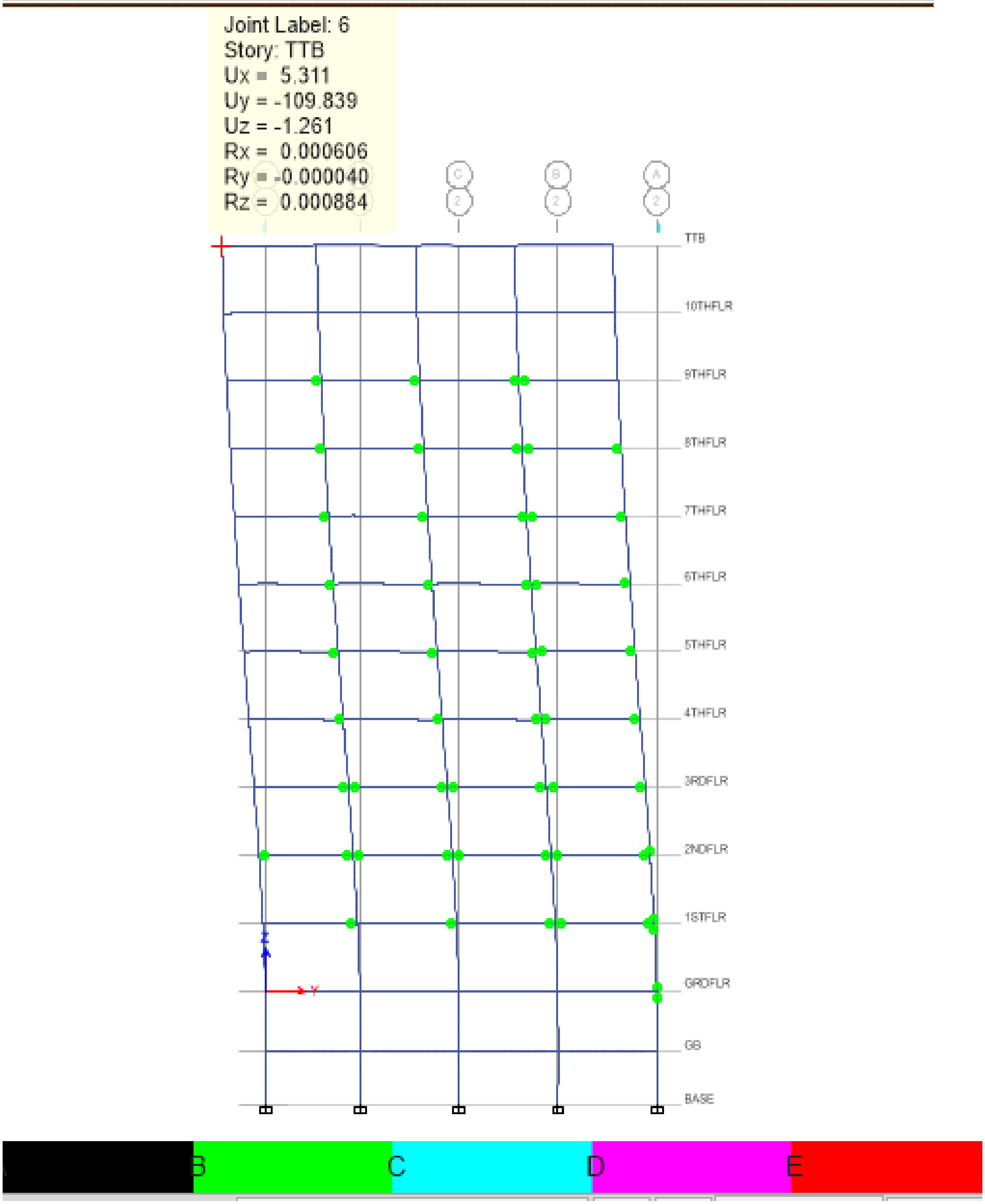
Tabulated Plot Coordinates

Capacity Curve Coordinates

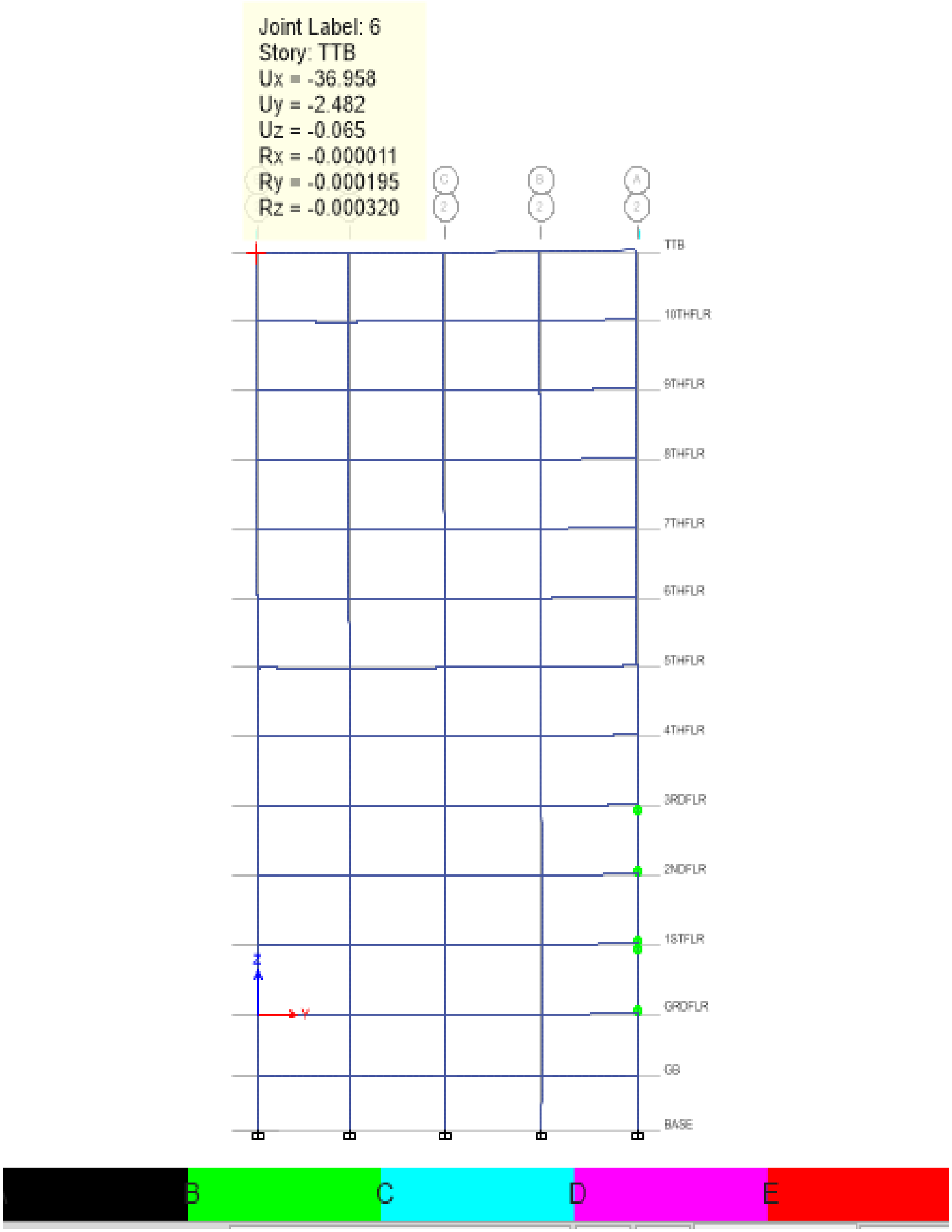
Sd Mm	**Sa, g**	**Period sec**
0	0	0.599
17.642	0.198093	0.599
25.482	0.28218	0.603
25.588	0.281132	0.605
26.324	0.288678	0.606
26.367	0.288502	0.607
26.378	0.2886	0.607
26.399	0.288354	0.607
26.716	0.291726	0.607
26.759	0.291771	0.608
26.762	0.291791	0.608
26.762	0.291679	0.608
26.851	0.292589	0.608



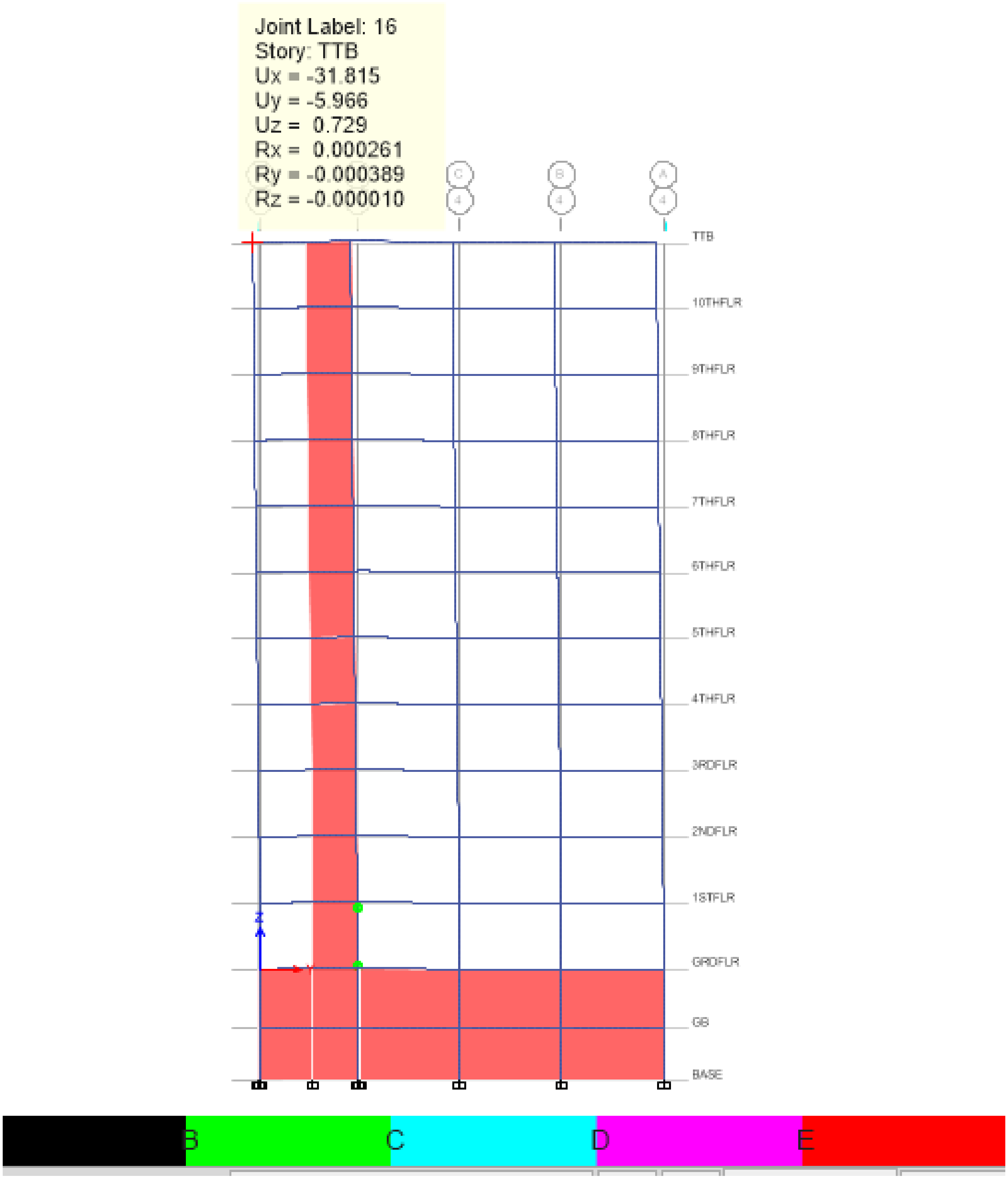
Elevation View G1-2 Displacements and hinge states(PushY) at Step 6 [mm]



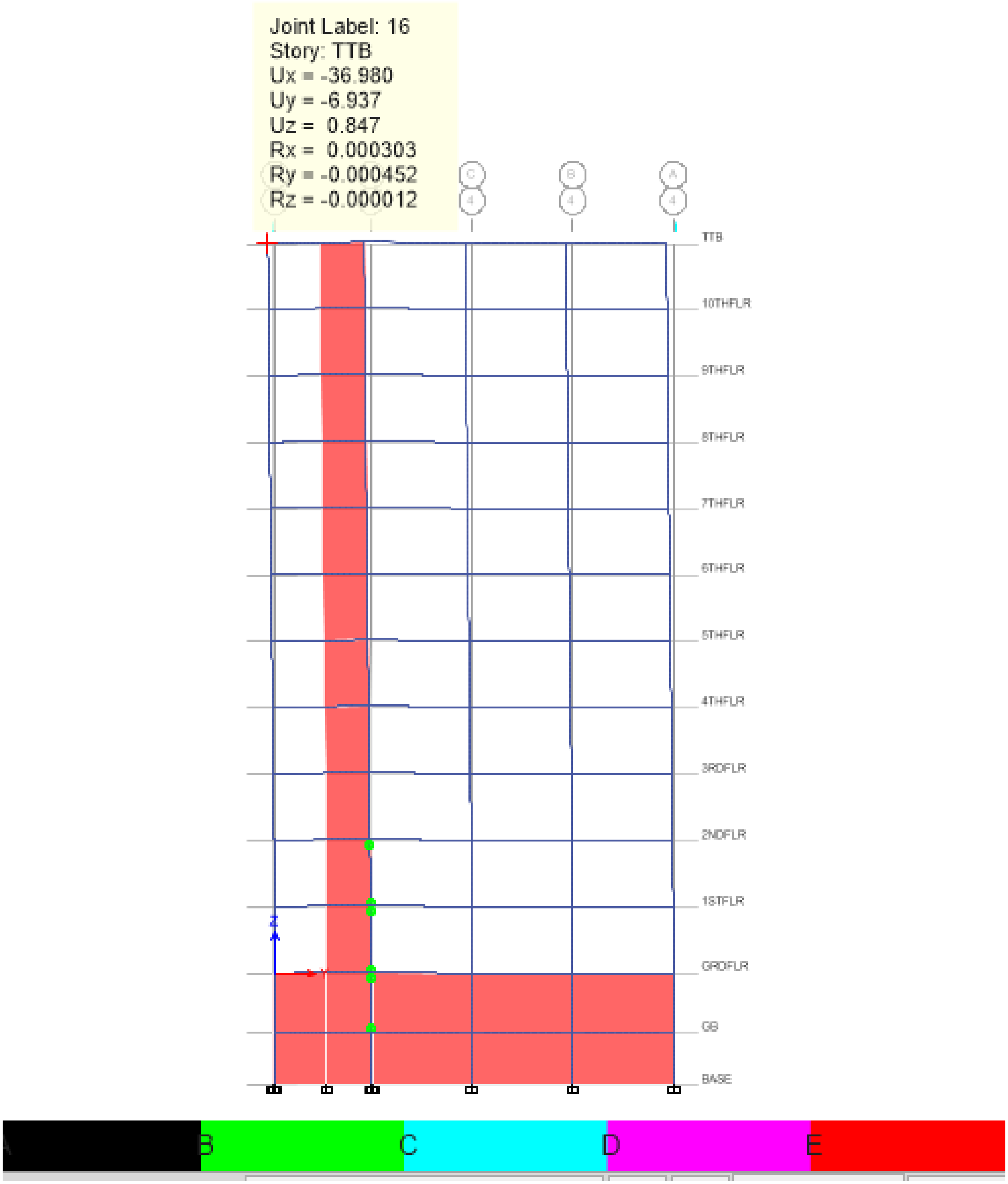
Elevation View G1-2 Displacements and hinge states (PushY) at Step 6 [mm]



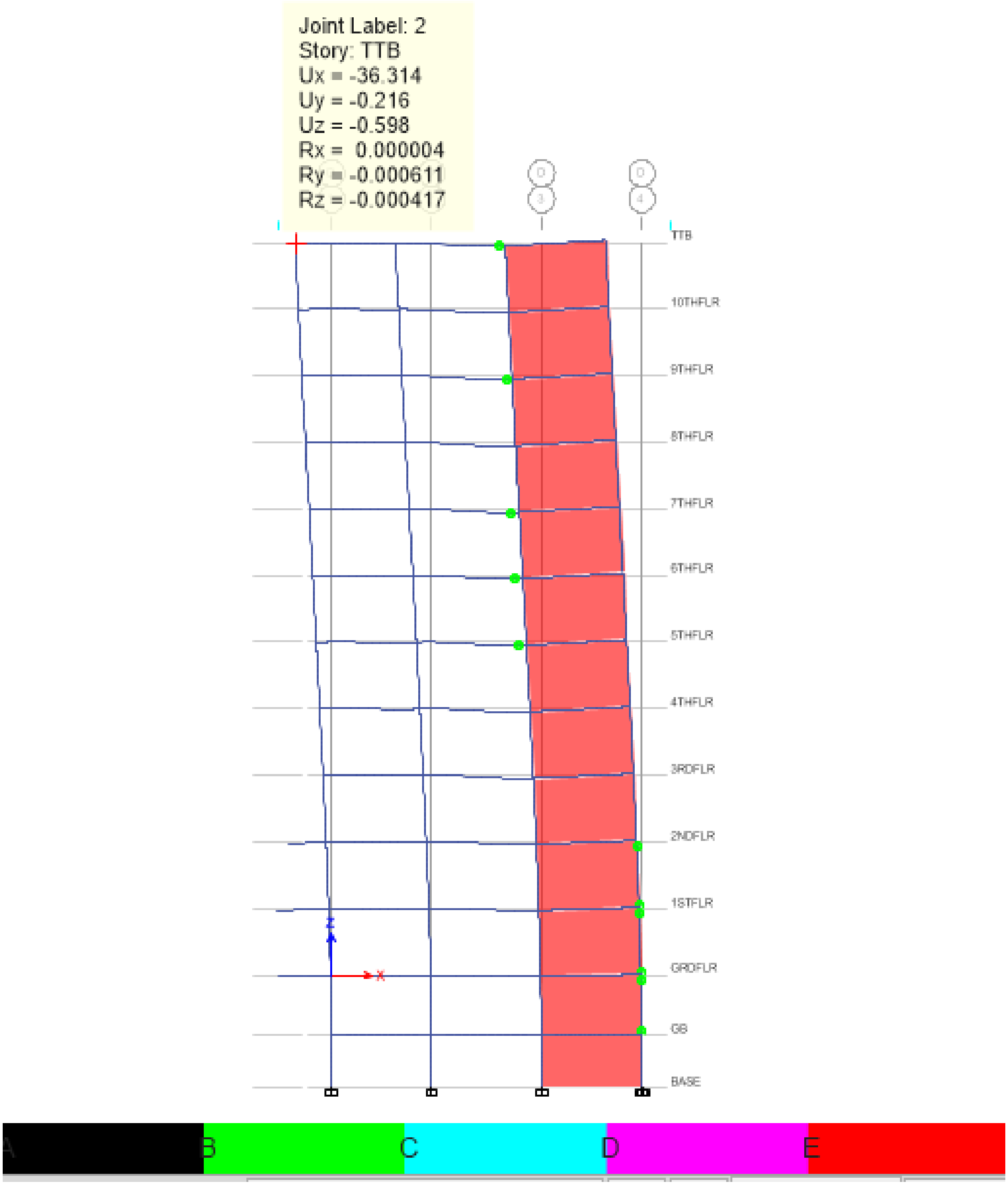
Elevation View G1-2 Displacements (PushX) and hinge states Step 9 [mm]



Elevation View G1-4 Displacements and hinge states (PushX) at Step 7[mm]



Elevation View G1-4 Displacements and hinge states (PushX) at Step 9 [mm]



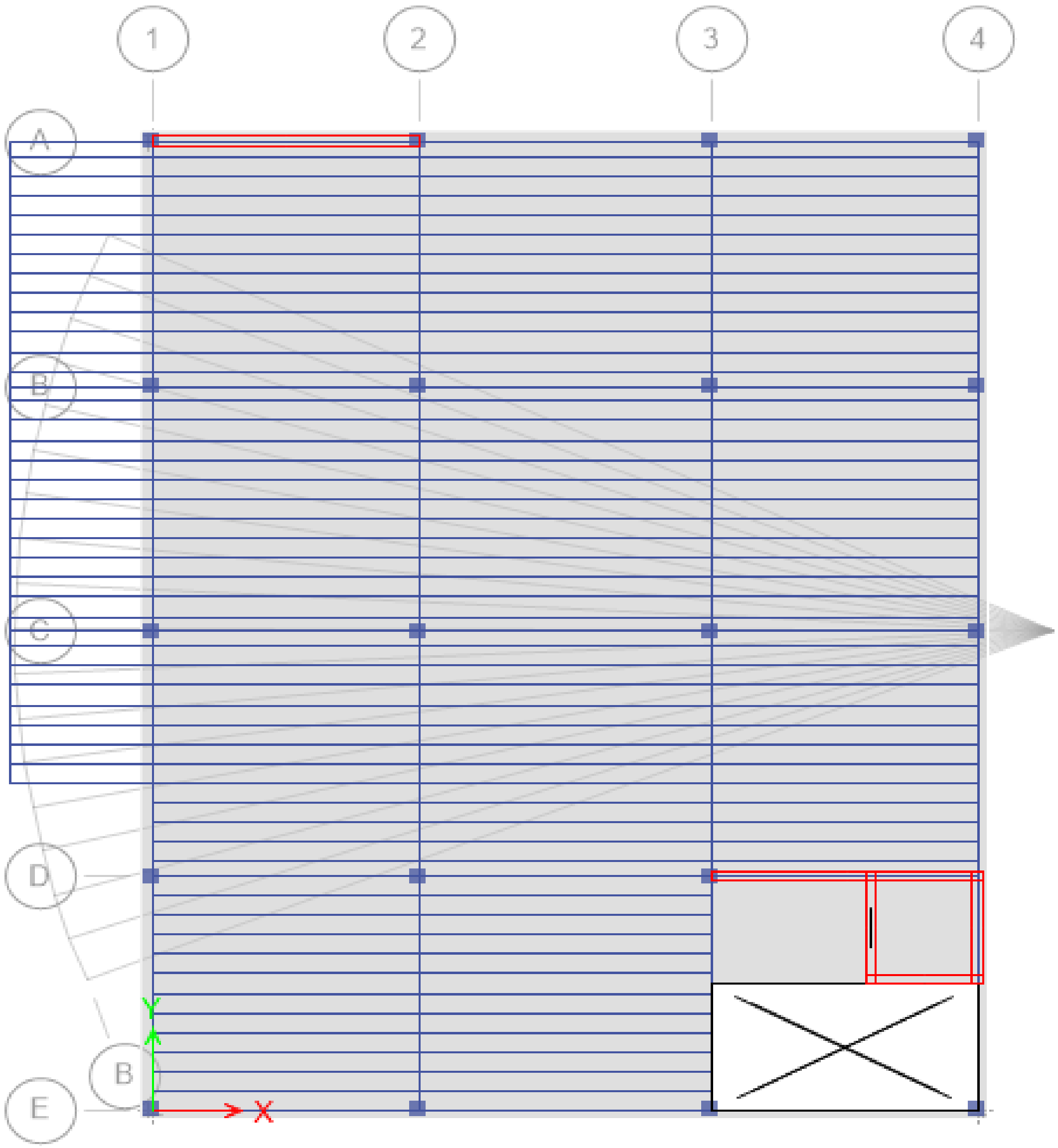
Elevation View G1-D Displacements and hinge state(PushX) Step 9 [mm]

TABLE: Hinge States

Story	Frame/Wall	Load Case/Combo	Assigned Hinged	Generated Hinge	M3, KN m	Hinge State	Hinge Status
TTB	B1	PushX Max	Auto M3	B1H1	0.7001	A to B	A to IO
TTB	B1	PushX Max	Auto M3	B1H2	0	A to B	A to IO
TTB	B1	PushX Max	Auto M3	B1H3	0.7001	A to B	A to IO
TTB	B1	PushX Max	Auto M3	B1H4	0	A to B	A to IO
TTB	B1	PushY Max	Auto M3	B1H1	0	A to B	A to IO
TTB	B1	PushY Max	Auto M3	B1H2	5.3591	A to B	A to IO
TTB	B1	PushY Max	Auto M3	B1H3	0	A to B	A to IO
TTB	B1	PushY Max	Auto M3	B1H4	5.3591	A to B	A to IO
10THFLR	B1	PushX Max	Auto M3	B1H5	0	A to B	A to IO
10THFLR	B1	PushX Max	Auto M3	B1H6	0.0867	A to B	A to IO
10THFLR	B1	PushX Max	Auto M3	B1H7	0	A to B	A to IO
10THFLR	B1	PushX Max	Auto M3	B1H8	0.0867	A to B	A to IO
10THFLR	B1	PushY Max	Auto M3	B1H5	0	A to B	A to IO
10THFLR	B1	PushY Max	Auto M3	B1H6	67.0538	A to B	A to IO
10THFLR	B1	PushY Max	Auto M3	B1H7	0	A to B	A to IO
10THFLR	B1	PushY Max	Auto M3	B1H8	67.0538	A to B	A to IO
9THFLR	B1	PushX Max	Auto M3	B1H9	0	A to B	A to IO
9THFLR	B1	PushX Max	Auto M3	B1H10	0.6097	A to B	A to IO
9THFLR	B1	PushX Max	Auto M3	B1H11	0	A to B	A to IO
9THFLR	B1	PushX Max	Auto M3	B1H12	0.6097	A to B	A to IO
9THFLR	B1	PushY Max	Auto M3	B1H9	0	A to B	A to IO
9THFLR	B1	PushY Max	Auto M3	B1H10	78.8999	B to C	A to IO
9THFLR	B1	PushY Max	Auto M3	B1H11	0	A to B	A to IO
9THFLR	B1	PushY Max	Auto M3	B1H12	78.8999	B to C	A to IO
8THFLR	B1	PushX Max	Auto M3	B1H13	0	A to B	A to IO
8THFLR	B1	PushX Max	Auto M3	B1H14	0.525	A to B	A to IO
8THFLR	B1	PushX Max	Auto M3	B1H15	0	A to B	A to IO
8THFLR	B1	PushX Max	Auto M3	B1H16	0.525	A to B	A to IO
8THFLR	B1	PushY Max	Auto M3	B1H13	0	A to B	A to IO
8THFLR	B1	PushY Max	Auto M3	B1H14	78.9112	B to C	A to IO
8THFLR	B1	PushY Max	Auto M3	B1H15	0	A to B	A to IO
8THFLR	B1	PushY Max	Auto M3	B1H16	78.9112	B to C	A to IO
7THFLR	B1	PushX Max	Auto M3	B1H17	0	A to B	A to IO
7THFLR	B1	PushX Max	Auto M3	B1H18	0.3464	A to B	A to IO
7THFLR	B1	PushX Max	Auto M3	B1H19	0	A to B	A to IO
7THFLR	B1	PushX Max	Auto M3	B1H20	0.3464	A to B	A to IO

Story	Frame/Wall	Load Case/Combo	Assigned Hinged	Generated Hinge	M3, KN m	Hinge State	Hinge Status
7THFLR	B1	PushY Max	Auto M3	B1H17	0	A to B	A to IO
7THFLR	B1	PushY Max	Auto M3	B1H18	78.0319	B to C	A to IO
7THFLR	B1	PushY Max	Auto M3	B1H19	0	A to B	A to IO
7THFLR	B1	PushY Max	Auto M3	B1H20	78.0319	B to C	A to IO
6THFLR	B1	PushX Max	Auto M3	B1H45	0	A to B	A to IO
6THFLR	B1	PushX Max	Auto M3	B1H46	0.1814	A to B	A to IO
6THFLR	B1	PushX Max	Auto M3	B1H47	0	A to B	A to IO
6THFLR	B1	PushX Max	Auto M3	B1H48	0.1814	A to B	A to IO
6THFLR	B1	PushY Max	Auto M3	B1H45	0	B to C	A to IO
6THFLR	B1	PushY Max	Auto M3	B1H46	78.2131	B to C	A to IO
6THFLR	B1	PushY Max	Auto M3	B1H47	0	B to C	A to IO
6THFLR	B1	PushY Max	Auto M3	B1H48	78.2131	B to C	A to IO
5THFLR	B1	PushX Max	Auto M3	B1H41	0	A to B	A to IO
5THFLR	B1	PushX Max	Auto M3	B1H42	0	A to B	A to IO
5THFLR	B1	PushX Max	Auto M3	B1H43	0	A to B	A to IO
5THFLR	B1	PushX Max	Auto M3	B1H44	0	A to B	A to IO
5THFLR	B1	PushY Max	Auto M3	B1H41	0	B to C	A to IO
5THFLR	B1	PushY Max	Auto M3	B1H42	78.1633	B to C	A to IO
5THFLR	B1	PushY Max	Auto M3	B1H43	0	B to C	A to IO
5THFLR	B1	PushY Max	Auto M3	B1H44	78.1633	B to C	A to IO
4THFLR	B1	PushX Max	Auto M3	B1H49	0	A to B	A to IO
4THFLR	B1	PushX Max	Auto M3	B1H50	0	A to B	A to IO
4THFLR	B1	PushX Max	Auto M3	B1H51	0	A to B	A to IO
4THFLR	B1	PushX Max	Auto M3	B1H52	0	A to B	A to IO
4THFLR	B1	PushY Max	Auto M3	B1H49	0	B to C	A to IO
4THFLR	B1	PushY Max	Auto M3	B1H50	78.6526	B to C	A to IO
4THFLR	B1	PushY Max	Auto M3	B1H51	0	B to C	A to IO
4THFLR	B1	PushY Max	Auto M3	B1H52	78.6526	B to C	A to IO
3RDFLR	B1	PushX Max	Auto M3	B1H21	0	A to B	A to IO
3RDFLR	B1	PushX Max	Auto M3	B1H22	0	A to B	A to IO
3RDFLR	B1	PushX Max	Auto M3	B1H23	0	A to B	A to IO
3RDFLR	B1	PushX Max	Auto M3	B1H24	0	A to B	A to IO
3RDFLR	B1	PushY Max	Auto M3	B1H21	0	B to C	A to IO
3RDFLR	B1	PushY Max	Auto M3	B1H22	78.8288	B to C	A to IO
3RDFLR	B1	PushY Max	Auto M3	B1H23	0	B to C	A to IO
3RDFLR	B1	PushY Max	Auto M3	B1H24	78.8288	B to C	A to IO
2NDFLR	B1	PushX Max	Auto M3	B1H25	3.7202	A to B	A to IO

Story	Frame/Wall	Load Case/Combo	Assigned Hinged	Generated Hinge	M3, KN m	Hinge State	Hinge Status
2NDFLR	B1	PushX Max	Auto M3	B1H26	0	A to B	A to IO
2NDFLR	B1	PushX Max	Auto M3	B1H27	3.7202	A to B	A to IO
2NDFLR	B1	PushX Max	Auto M3	B1H28	0	A to B	A to IO
2NDFLR	B1	PushY Max	Auto M3	B1H25	0	B to C	A to IO
2NDFLR	B1	PushY Max	Auto M3	B1H26	80.0801	B to C	A to IO
2NDFLR	B1	PushY Max	Auto M3	B1H27	0	B to C	A to IO
2NDFLR	B1	PushY Max	Auto M3	B1H28	80.0801	B to C	A to IO
1STFLR	B1	PushX Max	Auto M3	B1H29	5.38	A to B	A to IO
1STFLR	B1	PushX Max	Auto M3	B1H30	0	A to B	A to IO
1STFLR	B1	PushX Max	Auto M3	B1H31	5.38	A to B	A to IO
1STFLR	B1	PushX Max	Auto M3	B1H32	0	A to B	A to IO
1STFLR	B1	PushY Max	Auto M3	B1H29	0	B to C	A to IO
1STFLR	B1	PushY Max	Auto M3	B1H30	78.6815	B to C	A to IO
1STFLR	B1	PushY Max	Auto M3	B1H31	0	B to C	A to IO
1STFLR	B1	PushY Max	Auto M3	B1H32	78.6815	B to C	A to IO
GRDFLR	B1	PushX Max	Auto M3	B1H33	1.5199	A to B	A to IO
GRDFLR	B1	PushX Max	Auto M3	B1H34	0	A to B	A to IO
GRDFLR	B1	PushX Max	Auto M3	B1H35	1.5199	A to B	A to IO
GRDFLR	B1	PushX Max	Auto M3	B1H36	0	A to B	A to IO
GRDFLR	B1	PushY Max	Auto M3	B1H33	0	A to B	A to IO
GRDFLR	B1	PushY Max	Auto M3	B1H34	29.7743	A to B	A to IO
GRDFLR	B1	PushY Max	Auto M3	B1H35	0	A to B	A to IO
GRDFLR	B1	PushY Max	Auto M3	B1H36	29.7743	A to B	A to IO
GB	B1	PushX Max	Auto M3	B1H37	0	A to B	A to IO
GB	B1	PushX Max	Auto M3	B1H38	0	A to B	A to IO
GB	B1	PushX Max	Auto M3	B1H39	0	A to B	A to IO
GB	B1	PushX Max	Auto M3	B1H40	0	A to B	A to IO
GB	B1	PushY Max	Auto M3	B1H37	0.4463	A to B	A to IO
GB	B1	PushY Max	Auto M3	B1H38	2.5472	A to B	A to IO
GB	B1	PushY Max	Auto M3	B1H39	0.4463	A to B	A to IO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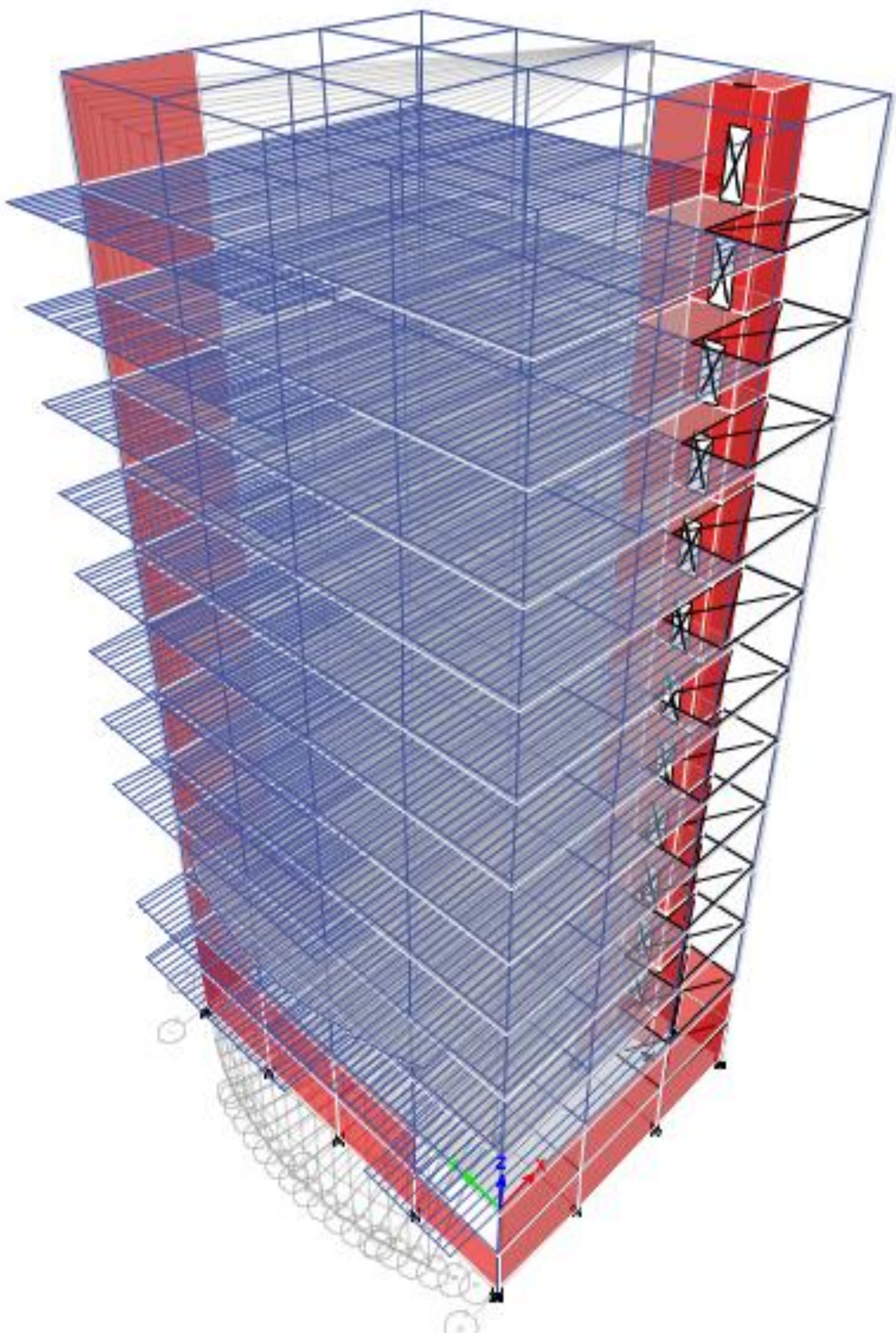

Structural Model of the building

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