Structural Needs of Existing Buildings in Gaza for Earthquake Resistance

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A ThesisSubmitted in partial Fulfillment of the Requirements for Degree of Master of Science in Structure

The Islamic University of Gaza- Palestine
March, 2009
إهداء

إلى من كانوا رمزا للعطاء.. وأي وأمي
إلى روح والدي رحمه الله
إلى إخوتي و زوجتي وأولادي
إلى زملائي وأساتذتي الأعزاء

الباحث

م. أشرف عبد الوهاب قنديل
Abstract:

This study aims at finding a quick and easy way to evaluate Gaza buildings for their resistance to earthquakes and make recommendations for strengthening the existing buildings, and finally to conduct recommendations for the local governorates for designing new buildings in Gaza.

To achieve this aim many evaluation methods used all over the world were studied, reviewed and tested on Gaza buildings.

In this research, a new method was developed to evaluate the buildings in Gaza Strip. The developed method classifies the Gaza buildings into sufficient, intermediate and insufficient resistance. In the intermediate resistance criteria, strengthening to the building is needed to make the building sufficient to earthquake resistance. This method has been developed by combination the Israeli method and Turkish method. The Israeli method is suitable to be used on Gaza buildings in terms of the similarity of geological situation, the construction methods and materials used in the two regions. It classifies building due to earthquake resistance only into sufficient and insufficient resistance. In the Turkish method a detailed calculation is made to classify the buildings into sufficient, intermediate and insufficient resistance. In order to adopt this method for modification, a detailed comparison is made between Palestine and Turkey through several aspects of the system such as soil, seismic zones, structural systems and construction quality.

In the developed method, the first step is to perform a rapid scanning to the building if the building is classified as insufficient a detailed computation is needed to reach the final evaluation result. If the final result is sufficient the building will be accordingly classified as to have enough resistance to earthquakes. If the result is intermediate, the building is classified as in need for strengthening. If the result was insufficient the building is classified accordingly, and the recommendation is to demolish it, or it needs major strengthening.

The developed approach is applied on thirty three different Gaza buildings which include: residential housing buildings, tower buildings, schools, health clinic and finally for asbestos shelters.

Finally, classification for the buildings according to earthquake resistance are made and recommendations for strengthen every kind of the buildings are proposed.

Based on this study, it was found that the structural system used on Gaza Strip which is Skelton type is an appropriate system to resist earthquakes of high intensity. The weakness of this system appeared in the case of the presence of soft story. Tower buildings are classified as intermediate and weak in resisting earthquakes, according to the area of shear walls in the building. The reinforced concrete frame system which is used in public buildings is suitable and adequate to resist earthquakes of high intensity. The asbestos buildings are weak and unsuitable in resisting earthquakes forces.
من الخص: 
تهدف هذه الدراسة إلى إيجاد طريقة سريعة وسهلة لتقديم مباني قطاع غزة لمقاومة الزلازل وتقييم توصيات لمقاومة المباني القائمة، وفي النهاية يتم تقديم توصيات للسلطات المحلية لتصديم المباني الجديدة في غزة. لتحقيق هذا الهدف تم دراسة العديد من طرق التقييم المستخدمة في كل إناء العالم، ومراجعتها واختيارها على مباني قطاع غزة.

في هذا البحث تم تطوير طريقة جديدة لتقديم مباني قطاع غزة وتقسيم الطريقة الجديدة مباني قطاع غزة إلى ملايين أو وسط أو غير ملائم للمقاومة. في حالة التقييم الوسيطي للمقاومة تتطلب هذه الحالة عمل تقييم للمبنى ليتحول إلى مناسب لمقاومة الزلازل. طورت هذه الطريقة من خلال دمج تقنيات إسرائيلية والتركية. تعتبر الطريقة الإسرائيلية مناسبة للاستخدام في قطاع غزة وذلك لتشابه الوضع الجيولوجي، وطرق البناء والمواد المستخدمة في المناطق، وهي تقيم المبنى من ناحية مقاومة الزلازل إلى ملائم أو غير ملائم للمقاومة.

تتم عمل حسابات مفصلة في الطريقة التركية لتصنيف المبني، أما ملائم أو وسط أو غير ملائم للمقاومة. ومن أجل اعتماد هذه الطريقة في التحديث تم عمل مقارنة مفصلة بين فلسطين وتركيا من عدة جوانب مثل التربة، وزلزالية المنطقة، والطبيعة الإنشائية وجودة البناء.

الخطوة الأولى في الطريقة المطورة هي القيام بمسح سريع للمبنى. فإذا كان التصنيف غير ملائم، يتم عمل حسابات مفصلة للتوصل إلى نتيجة التقييم النهائية. إذا كانت نتيجة التقييم هي ملائم يصنيف المبنى نهائيا بأنه له مقاومة مناسبة للزلازل. وإذا كانت تصنيف المبنى وسطي يصنف المبنى على أنه يحتاج إلى تقوية. وإذا كانت النتيجة غير ملائم فإن المبنى يصنف وفقا لذلك، وتوصي تكون أما بالهدم أو يحتاج إلى تقوية كبيرة.

تم تطبيق هذا الاقتراح على عدد ثلاثة وثلاثون من المباني المختلفة في غزة والتي تشمل: المباني السكنية ومباني أبراج ومدارس وعيادات صحية. وأخيرا بروت من الأسست.

في النهاية تم عمل تصنيف للمبنى يسمح لمقاومة الزلازل وتقييم توصيات من أجل تقوية كل نوع من المباني التي تم اقتصارها.

بناء على هذه الدراسة، وجد أن النظام الإنشائي المستخدم في قطاع غزة وهو نظام الأعمدة والجسم هو نظام مناسب لمقاومة الزلازل ذات الشدة العالية. يظهر الضعف في هذا النظام في حالة وجود الطواقي اللينة، عليه أن تصبح على أنها متوسطة أو ضعيفة في مقاومة الزلازل حسب مساحة حوالات القص في المبنى. نظام الإطارات الخرسانية المستخدم في المباني العامة هو ملائم. وكافيا لمقاومة الزلازل ذات الشدة العالية. أما مباني الأسست فهي ضعيفة وغير مناسبة في مقاومة قوى الزلازل.
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Abbreviations

$V_{ci}$  Shear capacity.
$V_y$  Base shear capacity.
n  Number of stories.
BCPI  Basic capacity index.
CPI  Capacity index.
$C_A$, $C_M$  Coefficients that reflect the architectural factures.
$C_{AS}$  Reflects the soft story feature.
$C_{ASC}$  Reflects short column.
$C_{AP}$  Reflects plan irregularity.
$C_{AF}$  Reflects frame irregularity.
SPI  Seismic priority index.
NSI  Non structural index.
SI  Structural index.
SR-1  The sub-rating.
SR-2  A second sub rating.
$S_o$  Basic score.
$\Delta S$  Modifiers.
$S$  The structural score.
$Z$  Peak ground acceleration.
(MNLSTFI) Minimum normalized lateral stiffness index.
(MNLSTI) Minimum normalized lateral strength index.
NRS  Normalized redundancy score.
SSI  Soft story index.
Or  Overhang ratio.
NRR  Normalized redundancy ratio.
Agf  Area of ground floor.
$D_{ILS}$  Damage index score corresponding to LSPC
$D_{IIO}$  Damage index score corresponding to IOPC.
$CF_{LSPC}$  Cut off value corresponding to LSPC.
$CF_{IOPC}$  Cut off value corresponding to IOPC.
$F_{a}$  Site coefficient.
$F_{v}$  Site coefficient.
$S_s$  Mapped spectral acceleration for short periods.
$S_1$  Mapped spectral acceleration for short periods.
$SD_s$  The design spectral response at short periods.
$SD_1$  The design spectral response acceleration at second periods.
I  Occupancy importance factor
R  Response modification factor
CMC  The cut of modification coefficients
$M_w$  Earthquake magnitude
\( V \)  Seismic base shear  
\( C_s \)  Seismic response coefficients  
\( W \)  Weight of structure and contents
CHAPTER "1"
Introduction

1.1 Introduction
Earthquakes are one of the greatest challenges to designers of buildings and other civil engineering structures. The potential for violent ground motions lasting not more than seconds to cause great destruction. Although the geographical regions of major hazard are generally well established, earthquakes may strike without warning. Taking seismic resistance into account from the early stages limits the impact on the costs of seismic provision, which are generally less than 10% of structural costs, and a much lower percentage of total cost. The major consequences of an earthquake arise from the violent ground motion it produces. The shaking is not life-threatening in itself, it is consequential collapse of structures that is the main cause of death, injury and economic loss. The primary natural hazards due to earthquakes can be summarized as follows:-

1- Collapse of buildings and other structures are the main source of death and injury.
2- Fall of objects e.g. ornaments, parapets, cladding from the outside of buildings, which also often proves lethal.
3- Explosions of gas and oil tanks and other dangerous chemicals.

The aims of earthquake resistant design are to prevent collapse during the worst credible event, to prevent structural damage and limit non-structural damage in earthquake expected to occur once or twice during the life time of a structure.

The structure shall include complete lateral and vertical force resisting systems capable of providing adequate strength, stiffness, and energy dissipation capacity to withstand the design ground motions within the prescribed limits of deformation and strength demand. The design ground motions shall be assumed to occur along any direction of the structure. The adequacy of the structural systems shall be demonstrated through construction of a mathematical model and evaluation of this model for the effects of the design ground motions. Individual members shall be provided with adequate strength to resist the shears, axial forces and moments determined and connections shall develop the strength of the connected members. The deformation of the structure shall not exceed the prescribed limits. The old buildings which are not designed for earthquake resistance is evaluated by the new approach to choose away of strengthen.

In this work, selected preliminary methods for evaluation are applied for some of public buildings like schools, clinics and some residential houses. The results are discussed and finally recommendations are established for every type of buildings used on Gaza Strip.

1.2 General objective
The main aim of this study is to develop an approach which can be used to classify all types of buildings with respect to earthquake resistant. This will enable all the people to reassure for their buildings and pushing the engineers to design the buildings for earthquakes resistant for all new buildings. All the old buildings which
classified as insufficient earthquake resistance buildings will find an easy way of strengthen.

1.3 Specific objectives
1- Developing a method for evaluation of earthquake resistance of existing buildings.
2- Applying the approach to public and residential buildings as a case study.
3- Making proper recommendations for strengthening each type of buildings.

1.4 Research problem
Some buildings in Gaza strip were designed and constructed without taking proper considerations for earthquake effects. Thus, such buildings need to be altered to enable and withstand earthquake effects.

It is necessary to classify existing buildings in accordance to their seismic force resistance and proposes strengthening techniques suitable for each type. Palestine is classified as an active seismic zone; it means that in any earthquake may strike this region many buildings will be affected.

History books show that, many earthquakes have struck Palestine, one happened in 1033 in Jericho and another happened in 1094 in Al Romla and another happened in 1927 in the north of Jericho with magnitude 6.25 Richter[1]. These cites are located on the centers of Earthquakes but Gaza was affected as well. Recently an Earthquake has stroke Cairo directly and Palestine was affected and five thousands of people died in Cairo.

1.5 Scope of study
This study aims to investigate many earthquake evaluation methods which are used all over the world. The researcher has developed a new method which will be used for evaluation Gaza concrete buildings. The main output of this study is a developed approach for earthquake evaluation for Gaza buildings.

1.6 Research methodology
Recent and old studies were reviewed about earthquake engineering and many evaluation methods which were published. Existing evaluation techniques are reviewed, their application to Palestine buildings is discussed, and a modified method for evaluation of buildings for seismic resistance is developed. The new approach is applied on different buildings in Gaza strip. From the results and the classification of buildings in respect to earthquake resistance, recommendations for strengthening buildings techniques are developed.

1.7 Research limitations and difficulties
This study is concerned only with concrete buildings. Structural steel buildings need further studies.
1.8 Structure of the thesis
This research consists of six main chapters as followings:-

Chapter 1 includes introduction to the subject, the main aims of research, research problem, research methodology, research limitations and difficulties and the structure of the thesis.

Chapter 2 includes a review of earthquake engineering and some evaluation methods used all over the world.

Chapter 3 includes classifications of Gaza Strip buildings in respect to building material, structural systems and buildings use.

Chapter 4 includes applying some evaluation methods on sample buildings from Gaza Strip, recommendations for each methods if it suitable to Gaza or not and developing a new approach.

Chapter 5 includes the developed Israeli method of evaluation, and the new approach which is proposed for Gaza buildings evaluation.

Chapter 6 includes the application of the new approach on samples of different buildings on Gaza Strip, and the results and comments are established.

Chapter 7 includes the achievement of applying the new approach, conclusions about the results and recommendations to strengthen every type of Gaza buildings.
CHAPTER "2"
Literature Review

In this chapter earthquake engineering and some evaluation method used in different parts of the world were discussed, help in develop a new approach to Gaza buildings.

2.1 Earthquakes

An earthquake is shaking of the earth caused by pieces of the crust of the earth that suddenly shift. The crust, the thin outer layer, is mostly cold and brittle rock compared to the rock deeper inside. The most common cause of earthquakes is faulting. A fault is a break in the earth’s crust along which movement occurs. The study of this movement is known as plate tectonics. There are three types of plate boundaries [2].

Spreading zones, transform faults and seduction zones. At spreading zones, molten rock rises, pushing two plates apart. Most spreading zones are found in oceans spreading zones usually have earthquakes at shallow depths (within 30 kilometers of the surface). Transform faults are found where plates slide past one another. Earthquakes at transform faults tend to occur at shallow depths and form fairly straight linear patterns. Seduction zones are found where one plate overrides, or subducts, another, pushing it down ward into the mantle where it melt. Seduction zones are characterized by deep ocean trenches, shallow to deep earthquakes, and mountain ranges containing active volcanoes [2].

The point beneath the earth’s surface where the rocks break and move is called the focus of the earthquake. The focus is the under ground point of origin of on earthquake. Directly above the focus, on the earth’s surface is the epicenter. Earthquake waves reach the epicenter first. During an earthquake, the most violent shaking is found at the epicenter. Earthquake waves are known as seismic waves. Scientists have learned much about earthquakes and the interior of the earth by studying seismic waves. There are three many types of seismic waves. Each type of wave has characteristic speed and manner of travel [3].

2.1.1 Primary waves

Seismic waves that travel the fastest are called primary waves or P waves arrive at a given point before any other type of seismic wave. P waves travel through solids, liquids and gases. P waves are push - pull waves – As P waves travel; they push rock particles into the particles ahead of them, thus compressing the particles. The particles move back and forth in the direction the waves are moving [4].

2.1.2 Secondary waves

S waves arrive at a given point after P waves do. S waves travel through solid but not through liquids and gases. It does not change volume of particles like P waves but only the shape. The rock particles move at right angles to the direction of the waves [5].

2.1.3 Surface waves

The slowest moving seismic waves are called surface waves, or L waves originate at the epicenter surface waves travel along the surface of the earth, rather
than down into the earth. Although they are the slowest of the entire earthquake, L waves usually cause more damage than P or S waves [5].

2.2 Measuring earthquakes

The severity of an earthquake can be expressed in several ways. The magnitude of an earthquake, usually expressed by Richter scale is a measure of the amplitude of the seismic waves. The moment magnitude of an earthquake is a measure of the amount of energy released, an amount that can be estimated from seismograph reading. The intensity, as expressed by Modified Mercalli scale, is a subjective measure that describes how strong shock was felt at a particular location. The Richter scale, named after Dr. Charles F. Richter is the best known scale for measuring the magnitude of earthquakes. This scale is logarithmic an earthquake of magnitude Z is the smallest quake normally felt by people. Earthquakes with a Richter value of 6 or more or commonly considered major, great earthquakes have a magnitude of 8 or more on the Richter scale. The Modified Mercalli scale expresses the intensity of earthquakes effects in values ranging from I to XIT. The most commonly used adaptation converts the range of intensity from the condition of (I) not felt except by a very few under especially favorable conditions, to (XIT) damage total. Evaluation of earthquake intensity can be made only after eye witness reports and results of field investigations are studied and interpreted. Earthquakes destructiveness depends on many factors, in addition to magnitude and the local geologic conditions, these factors include the focal depth, the distance from the epicenter, and the design of buildings and other structures. The extent of damage also depends on density of population and construction in the area shaken by the quake [6].

2.3 Structural systems for resisting earthquakes

There are many structural systems used all over the world like [7]:

- Moment resisting frames.
- Grid frames.
- Shear walls.
- Block work infill.
- Frame – wall or dual systems.
- Coupled shear walls.
- Column and diaphragm system.

2.3.1 Moment – resisting frames

Moment – resisting frames derive their lateral strength - not from diagonal bracing members, but from the rigidity of the beam – column connections. They consist solely of horizontal beams and vertical columns. This system is used in Gaza Strip in schools, hospitals and mosques. It is considered a good system for resisting earthquake. The advantages of using moment – resisting frames to provide seismic resistance are as follows:

a) They provide a potentially highly ductile system with good degree of redundancy, which can allow freedom in the architectural planning of internal spaced and external cladding.

b) Their flexibility and associated long period may serve to detune the structure from the forcing motions on stiff soil or rock sites. The potential problems associated with moment – resisting frames are:
1. The beam – column joint region represents an area of high stress concentration which needs special skills to design successfully.

2. The low stiffness of moment – resisting frames tends to cause high story drifts, which may lead to unacceptable damage to cladding and other non structural elements and to other serious structural problems.

2.3.2 Grid frames

These comprise a uniform grid of frames in both directions, and are common in Japanese practice in low – to medium rise construction they are highly redundant and achieve a good spread of resistance to seismic forces both within the superstructure and to the foundations[4]. They have very good torsion resistance and coupled lateral, torsion is unlikely to be a problem, even with irregular plan shapes. The major disadvantages are that all the columns have to be designed for biaxial loads. All beams and columns have to be designed and detailed for ductility. This system is used in big halls and theaters.

2.3.3 Shear walls

Shear walls are more rationally known as structural walls in new buildings, since their flexural behavior is usually more important than their shear behavior. They avoid the stress concentrations found at the beam – column joint regions of frames, and avoid some of the dependence on good form work and steel fixing skills associated with frames. Considerable ductility is possible in slender shear walls which reach their ultimate strength in flexure before shear. The unfavorable features are firstly planning ones, walls present barriers which may interfere with architectural and services requirements. Secondly, lateral load resistance in shear wall buildings is usually concentrated on a few walls rather than on a large number of columns shear walls on their own are a highly suitable solution for medium – rise buildings up to about ten stories. All the tower buildings on Gaza Strip are using shear walls[5].

2.3.4 Block work infill

Rigid block work infill of external moment – resisting frames provides a good solution for providing thermal and acoustic insulation and weatherproofing. The block work infill causes a large increase in strength and stiffness, at the expense of a large reduction in ductility. If the infill is not uniform across the building, unsafe conditions such as the creation of a weak storey can result. Rigid infill with unreinforced blocks results in the creation of unpredictable and potentially lethal systems. Laboratory tests show that with appropriate design, reinforced block work infill can provide satisfactory resistance. This system is not used on Gaza Strip.

2.3.5 Frame wall or dual systems

Combinations of moment – resisting frames with shear walls are known as frame wall or dual systems. This combination can be structurally efficient and is favored in both US and Japanese practice as providing good redundancy. One advantage of frame – walls systems is that the shear wall can be used to prevent a weak storey forming in the moment resisting frame. This means that the relative's strength requirements to ensure a strong column / weak beam frame may theoretically be relaxed. This gives more freedom in selecting beam and column sizes and there is less concern about the strengthening effect that floor slabs have on beams [8].
2.3.6 Coupled shear walls

Coupled shear walls consist of two or more walls linked by horizontal coupling beams. The beams are often formed as a result of openings required through the wall at each floor level; the resulting structure becomes effectively a frame with very strong columns and weak beams. Most of the yielding is therefore confined to the coupling beams; provided they are adequately designed, which often involves use of diagonal steel, excellent ductility can be obtained, with good stiffness. Redundancy is also good, in that plastic energy dissipation is distributed between a numbers of coupling beams. Coupled shear walls have been used for medium-rise construction in New Zealand [5]. There is little field evidence as to whether their theoretical appeal translates in practice into superior performance during earthquakes.

2.4 Evaluation methods

Occurrences of recent earthquakes in all parts of the world and the resulting losses, especially human lives, have high lighted the structural inadequacy of buildings in terms of seismic resistance. The objective of these methods is to review various methods on seismic evaluation of existing building from different countries. It is expected that this comparative assessment of various evaluation methods will help identify the most essential components of a new evaluation method for use on Gaza Strip, which is not only reliable but also easy to use.

2.4.1 Preliminary Turkish method

A preliminary Turkish procedure is to assess the likely seismic performance of existing reinforced concrete building rapidly. In this procedure a capacity index is computed considering the orientation, size and material properties of the components comprising the lateral load resisting structural system [9]. This index is modified by several coefficients that reflect the quality of workmanship, materials and architectural features. The method classifies the buildings either as safe, meaning the building might suffer no severe damage or as unsafe, indicating that life safety performance level would not be met. The procedure has been tested and calibrated based on data compiled from damage surveys conducted after the earthquakes that occurred within the last decade in Turkey. In addition to what is collected from the street survey data on the size and orientation of the structural components, material properties and layout are needed. This procedure does not rely on sophisticated and time consuming analysis of the building but some quick calculations are performed. The procedure is recommended for low to medium rise reinforced concrete frame buildings with and without shear walls. So it can not used widely on Gaza Strip.

The basic premise is to approximate the base shear capacity of the building using the ground floor dimensions, size, orientation and concrete strength of the components comprising the lateral load resisting system. For this reason, the shear capacity of each structural component is computed based on concrete contribution using Eq (2.1).

\[
V_{ci} = c_a f_{ck} b_w h
\]

\[ (2.1) \]

\( V_{ci} \) stands for shear capacity of a rectangular concrete member with dimensions \( b_w \) and \( h \), and the direct concrete tensile strength of \( f_{ck} \). The coefficient \( c_a \) represents the combined effect of strength reduction factor and the empirical coefficient that relates shear strength to the tensile strength, and depend on specific design code. The total concrete shear capacity \( V_{ci} \) is obtained by adding up the capacities of individual...
members in the direction of each principal axis. In estimating the strength from visual in section, regional practice need to be taken into account for Turkey, the values indicated in Table (2.1) are recommended, based on the experience and common construction practice.

**Table 2.1  Recommended concrete strength values based on concrete quality**

<table>
<thead>
<tr>
<th>Concrete quality (From visual inspection)</th>
<th>Recommend compressive strength (fck) Mpa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Poor</td>
<td>&lt;10</td>
</tr>
<tr>
<td>Average</td>
<td>10-16</td>
</tr>
<tr>
<td>Good</td>
<td>&gt;16</td>
</tr>
</tbody>
</table>

The base shear capacity is given by Eq. (2.2).

\[
V_y = \frac{V_{ci}}{0.951^{0.125}} \quad (2.2)
\]

\(V_y\) stands of base shear Capacity and \(n\) stand of number of stories, Eq (2.2) is valid and applicable to buildings that have no infill walls contributing to their lateral load resistance. In the procedure, however, this effect has been quantified by increasing the yield base shear normalized with the total (\(A_{tf}\)) this has been accomplished by analyzing all selected building with and without inclusion of filler walls to develop a relationship between the yield base shear capacity with infill walls and \(V_{yw}\) without in fill walls (\(V_y\)).

The relation is recommended for reinforced concrete buildings in Turkey.

\[
V_{yw} = V_y \left( \frac{46 A_{nw}}{A_{tf}} + 1 \right) \quad (2.3)
\]

Once the yield base shear capacity of the building is estimated using Eqs (2.1) – (2.3) and index named Basic capacity Index (BCPI) is computed from Eq. (2.4)

\[
BCPI = \frac{V_{yw}}{V_{code}} \quad (2.4)
\]

Eq (2.4) is obtained by normalizing the yield base shear capacity with the elastic design base shear of the associated design code through which the seismic risk due to location and soil condition is also taken into account. Using the total area of the filler walls BCPI computed for an existing building can be used to assess the expected seismic performance if the building is assumed to be built according to specifications and possess no architectural features that would have negative effect on its performance.

\[
CPI = C_A C_M BCPI \quad (2.5)
\]

Where \(C_A\) and \(C_M\) are Coefficients that reflect the architectural features and Construction quality. Respectively the coefficient \(C_A\) is determined by subtracting several other coefficients from the base value of unity as indicated in Eq. (2.6)

\[
C_A = 1.0 - \left( C_{AS} + C_{ASC} + C_{AP} + C_{AF} \right) \quad (2.6)
\]

Following the conceptual description given above the steps involved in the procedure are summarized below:
**Step 1:** For the ground floor, compute the total concrete base shear capacity, $V_C$, using Eq. (2.1). If concrete strength could not be obtained via tests then use the values given in Table (2.1).

**Step 2:** Calculate estimated yield base shear capacity, $V_y$ from Eq. (2.2)

**Step 3:** Using the total area of infill masonry walls without openings and total floor area of the building, compute yield base shear capacity of the building with infill walls $V_{yw}$ from Eq. (2.3).

**Step 4:** Divide $V_{yw}$ with the code required design base shear, $V$ code computed according to the seismic design code of practice to obtain the BCPI.

**Step 5:** by visual inspection of the building and review of its drawing, determine the modification factor $C_A$ using Eq (2.6).

**Step 6:** Rank the building for the construction and workmanship quality based on the visual inspection and determine the coefficient $C_M$

**Step 7:** Compute the CPI using Eq. (2.5).

**Step 8:** Determine whether the building is expected to be safe or unsafe.

### 2.4.2 Canadian evaluation method [10-12]

In Canada, a building would generally go through a three-step process to address seismic hazard, namely screening, evaluation, and mitigation. Screening helps prioritize buildings such that buildings with the highest risk scores would warrant a more detailed analysis while buildings with the lowest risk scores may be exempted or deferred from further investigation. This detailed analysis determines if and to what extent a building needs strengthening. Fig (2.1) illustrates the seismic hazard mitigation procedure on screening (Step 1), evaluation (step 2), and mitigation (step 3) for buildings. This method is dependent on factors found for Canada, so it cannot be used without modifications for Gaza Strip buildings.

---

**Fig (2.1) Seismic mitigation procedure on screening, evaluation, and upgrading.**
2.4.2.1 Screening
Buildings can be screened using a risk management tool developed by National Research Council of Canada. Buildings are screened according to the building location, soil conditions, type and the use of structure, obvious building irregularities, the presence or absence of non-structural hazards, building age, occupancy characteristic and building importance. Screening can be used to numerically establish a seismic priority index (SPI) which results from the addition of a structural index (SI) and a non-structural index (NSI).

2.4.2.2 Screening parameters
The methodology of screening manual is based on:
1- Identifying the main features of the building, its location, occupancy, etc.
2- The individual numerical factors associated with the parameters as identified in (1),
3- The combined risk index which is essentially the mathematical product of these individual numerical factors.

Parameters considered in the screening process include (Fig 2.2).
- A: Seismicity.
- B: Soil Conditions.
- C: Type of structure.
- D: Building irregularities.
- E: Building importance (occupancy)
- F: Non-structural hazards.

The seismicity effect is determined by the location of building and applicable NBCC as given in Table (B.1) in appendix (B). The seismicity of a location is determined by the effective seismic zone, which was defined in the NBCC 1990[12]. The seismic parameter (A) can have a value between 1.0 and 4.0. The effect of the type of structure is determined by the type of structural system of the building and the applicable NBCC as given in Table (B.2) in appendix (B).

The effect of building importance is determined by the type and density of occupancy of the building and by the applicable NBCC as given in Table (B.3) in appendix (B). The building importance parameter considers post disaster buildings and special operational requirements. Depending on the occupancy type and density of the building, the building importance parameter (E) has a value of between 0.7 and 3.0.

2.4.2.3 Seismic priority index
The scoring system is made up of a structural index (SI) and a non-structural index (NSI) SI is related to possible risk to the building structure and NSI is related to the risk of non-structural building components.

The structural index, SI, is calculated as follows:
$$SI = A \times B \times C \times D \times E$$

Where A, B, C and E account for effects of seismicity, soil conditions, type of structure, building irregularities and building importance as shown in Fig 2.2. The non structural index, NSI is calculated as follows:
$$NSI = B \times E \times F$$

Where: F is the maximum value between $F_1$ for falling hazards to life and $F_2$ for hazards to vital.
2.4.2.4 Seismic hazard mitigation for buildings

- **Seiziemisty; A=1.0 to 4.0**
  - Effective Seismic Zone

- **Soil Conditions, B=1.0 to 2.0**
  - Rock, Stiff Soil, Soft Soil, liquefiable Soil, Unknown Soil

- **Type of Structure c=1.0 to 3.5**
  - Wood, Steel, concrete, precast, Masonry infill, Masonry

- **Building Irregularities D=1.0 to 4.0**
  - Vertical, Horizontal, short column, soft story, Pounding, Modifications

- **Building Importance, E= 0.7 to 3.0**
  - Occupancy and Operational Requirements

- **Structural Index "SI" = ABCDE**

- **Non-Structural Hazard F=Larger of F1 and F2**
  - Falling Hazards to life, $F_1=1.0$ to 6.0
  - Howard's to Vital Operations, $F_2=1.0$ to 6.0

- **Non Structural Index (NSI) = BEF**

- **Seismic Priority Index (SPI) = SI + NSI**

**Fig 2.2 Screening procedure**

The seismic priority index, SPI is equal to the sum of structural index and non-structural Index, i.e. SPI = SI + NSI as illustrated in Fig 2.2. The seismic priority index is related to the seismic risk for a building as per the NBCC 1990 requirements. The screening manual suggests that the potential seismic risk for a building is low with a SPI less than 10 medium with a SPI between 10 and 20 and high with a SPI higher than 20 (Fig 2.1).

Building with SPI scores of greater than 30 can be considered high risk and that an immediate assessment of the seismic performance of the building is required.

2.4.3 Long Beach evaluation method.

In 1971, the J.H. Wiggins company studied the earthquake safety of older buildings in the city of Long beach California[13]. A grading system was developed to assess the hazard of existing building, using death risk as the criterion. The system is a simple field and document checking procedure carried out by professional engineers with back ground in earthquake design, as much engineering judgment is required. Building is graded on a scale of 0 to 180 points, defining three degrees of hazard.

1) Less than 50 points constitutes a low degree of hazard, and no rehabilitation is necessary.
2) Between 50 and 100 points indicates an intermediate hazard, with some strengthening required to make the building safe.

3) More than 100 points classify the building as a serious life hazard, requiring demolition or major strengthening.

The grading system is broken down into five categories, with points assigned to each category as outlined in (Tables 2.2) which are framing system and/or walls (0 to 40 points), diaphragm and/or bracing system (0 to 20 points), partitions (0 to 20), special hazards such as building layout or soil condition (0 to 50 points). Wiggins report recommends guidelines for the extent of strengthening to be required. Normal repairs would prevent the collapse of the entire building, including walls. Minimum repairs prevent the collapse of the roofs and floors, but could allow masonry walls to collapse providing they do not overhang another building or adjoin a public way. Buildings undergoing only minimum repairs would limit to a life of five years, and a normal occupancy no more than five person years per year.

Table (2.2) Grading system of Long Beach evaluation method

<table>
<thead>
<tr>
<th>LOW (0-POINTS)</th>
<th>INTRMEDIATE (20 – POINTS)</th>
<th>HIGH (40 – POINTS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>* Reinforced concrete and steel vertical and lateral resisting frames adequately designed, with reinforced concrete and masonry filler walls.</td>
<td>* Reinforced concrete and steel vertical load frames</td>
<td>* Unreinforced masonry filler and bearing walls with poor quality mortar.</td>
</tr>
<tr>
<td>* Steel rigid frames adequately braced.</td>
<td>* Wood frames over three stories.</td>
<td></td>
</tr>
<tr>
<td>* Well anchored continuous double sheet metal decking.</td>
<td>* Unreinforced concrete and masonry filler walls, good mortar.</td>
<td></td>
</tr>
<tr>
<td>* Well anchored blocked plywood.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>* Well anchored steel bracing system other than rods.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Diaphragm and/or Bracing System

<table>
<thead>
<tr>
<th>LOW (0-POINTS)</th>
<th>INTRMEDIATE (20 – POINTS)</th>
<th>HIGH (40 – POINTS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>* Signal sheet metal decking high weight fills unblocked plywood diagonal sheathing and gypsum all anchored and adequate as diaphragms.</td>
<td>* Precast concrete units without fill anchored and adequate as diaphragms.</td>
<td>*Straight or diagonal wood sheathing without adequate connections to walls and no special nailing Incomplete or inadequate bracing systems.</td>
</tr>
<tr>
<td>* Well anchored reinforced concrete slabs and fills adequately as diaphragms.</td>
<td>* Diagonal rod bracing systems anchored and adequate design.</td>
<td></td>
</tr>
<tr>
<td>* Well anchored continuous double sheet metal decking.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>* Well anchored blocked plywood.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>* Well anchored steel bracing system other than rods.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### Table (2.2) Partitions

<table>
<thead>
<tr>
<th>LOW 0-POINTS</th>
<th>INTRMEDIATE (10 – POINTS)</th>
<th>HIGH (20 – POINTS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>* Many wood or metal stud bearing.</td>
<td>* Few unreinforced masonry, poor mortar.</td>
<td>* Many unreinforced Masonry, poor mortar and few steel or wood stud; bearing or non – bearing.</td>
</tr>
<tr>
<td>* Many reinforced masonry bearing.</td>
<td>* Moderate amount of wood and steel, bearing type.</td>
<td></td>
</tr>
<tr>
<td>* Moveable metal or gypsum board.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>* Few reinforced masonry non – bearing.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>* Few unreinforced masonry good quality mortar and well anchored.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Special hazard:

<table>
<thead>
<tr>
<th>LOW (0-5-10 POINTS)</th>
<th>INTRMEDIATE (10-15-20 – POINTS)</th>
<th>HIGH (20-35-50 POINTS)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>* Lack of symmetrical bracing or shear walls.</td>
<td>* Overhanging adjacent unreformed masonry filler or bearing walls poor quality mortar.</td>
</tr>
<tr>
<td></td>
<td>* Excessive length to width and height ratios (greater than 4; 1).</td>
<td>* Poor soil conditions uncomplicated &amp; saturated fills. Unstable side hill conditions</td>
</tr>
<tr>
<td></td>
<td>* Questionable soil conditions which could result in settlement or amplified ground motion in an earthquake.</td>
<td>* Inadequately anchored roof tanks or signs.</td>
</tr>
<tr>
<td></td>
<td>* Poorly anchored chandeliers and light fixtures</td>
<td>* Unreinforced masonry chimneys poor quality mortar.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>* Inadequately anchored ornamentation and veneer above 1st story.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Unbraced wood cripple stud walls in type v.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>*Non-bearing masonry walls parapet walls or appendages.</td>
</tr>
<tr>
<td></td>
<td>* Minor cracks</td>
<td>* Serious settlement or cracking.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>*Serious cracking, bowing or leaning of walls.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>*Minor un repaired earthquake damage.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>* Serious deterioration of structural materials.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>*Serious un repaired earthquake damage.</td>
</tr>
</tbody>
</table>

### Notes:
1- " Adequately " means conforming to current code.
2- Reinforcing of Concrete and masonry must comply with current code to be placed in "low" or "intermediate" categories.
3- Core test of masonry must be 750 PSI or greater to qualify.
4- Core test of masonry is below 750 PSI.
5- "Many" and "few" are relative terms. Typical, small offices and apartments would be considered to have "many" warehouse and garages to ordinarily.
6- Generally, expansive or relatively loose, silty or clayish soils refer to soils factor in section 2904 of 1970 UBC code.
7- Excessive wall openings include percentages of 50 and over for any one wall in any story. Excessive wall heights include height to thickness ratios of 30 or more.
8- Minor means few small cracks, generally less than 1/16 across.
9- Moderate means many cracks 1/16 to 1/8 across.
10- Serious means many cracks over 1/8 " across" settlements more than normal design limits.
11- Serious bowing or leaning more than 1/2 wall thickness.

2.4.4 National Bureau of standards field evaluation method (NBS).

The NBS uses a numerical grade to rate the earthquake safety of a building [14]. The NBS method can be applied to brace and unbraced steel frames, concrete frames, shear wall structures, bearing wall structures, and long-span roof structures. The tools for evaluation are plans and/ or a site inspection. The first step in the method to assign, modified Metrically Intensity (MMI) to the site, then a detailed data, collection form, (provided by NBS), is filled out, by visiting the site and inspection the plans, if available. The data provides the basis for rating the structural system with a numerical rating from 1 to 4 and the nonstructural ones with a letter grade. A general rating (GR) is assigned based on the type of framing, as moment – resists frames and braces frames receiving 1, unreinforced masonry walls 4.

The symmetry rating (S) is based on the eccentricity of the center of mass and the center of stiffness of the vertical resisting elements. The number of vertical resisting elements determines the quantity rating (Q) a combined symmetry and quantity rating (SQR) is taken as the average of S and Q.

The present condition (PC), based on observations of visible damage, is combined with the SQR in a weighted average yielding the sub-rating Eq2.7

\[ SR1 = \left( \frac{SQR + 2PC}{3} \right) \]  

(2.7)

The horizontal resisting elements are evaluated using a special form, arigidity factor (R) is assigned based on the flexibility of diaphragms. There is an anchorage rating (A) indicating how well the floors and roof are connected to the frame. The presence of chord (C) constitutes the third part of the horizontal element rating. A second sub-rating (SR2) is taken as the largest of R, A, and C. Special forms evaluate the nonstructural elements on a qualitative scale (Table B.4) in appendix (B).

A= Good, B= Fair, C= Poor, and X= Unknown.

The items rated are exit corridor and stair enclosure walls, other partitions, glass breakage, exterior appendages, gas leakage, ceilings, and light fixtures.

The basic structural rating

\[ \text{Basic Structural Rating} = GR + \frac{2(Largest \ of \ SR1 \ and \ SR2)}{3} \] 

(2.8)

The capacity ration (CR) is determined by dividing the basic structural rating by intensity level factor, which depends on modified Mercalli Scale.
CR= Basic Structural rating / Intensity level factor. The building is rated as Good (CR less than 1), Fair (CR between 1.0 and 1.40), poor (1.5 to 2.0) or very poor (over 2.0). Table (2.3) illustrates grading system of NBS method.

**Table (2.3) Grading system of NBS method**

<table>
<thead>
<tr>
<th>Type</th>
<th>General Rating GR</th>
</tr>
</thead>
<tbody>
<tr>
<td>A: Steel moment resistant frame</td>
<td>1</td>
</tr>
<tr>
<td>B: Steel frames – moment resistant capability unknown</td>
<td>2</td>
</tr>
<tr>
<td>C: Steel moment resistant frame</td>
<td>1</td>
</tr>
<tr>
<td>D: Concrete frames– moment resistance capability unknown</td>
<td>2</td>
</tr>
<tr>
<td>E: Masonry shear walls – unreinforced</td>
<td>4</td>
</tr>
<tr>
<td>F: Masonry or concrete shear walls – unreinforced</td>
<td>1</td>
</tr>
<tr>
<td>G: Combination unreinforced shears walls and moment resistant frames</td>
<td>2</td>
</tr>
<tr>
<td>H: Combination-reinforced shear walls and moment resistant frames</td>
<td>1</td>
</tr>
<tr>
<td>I: Braced frames</td>
<td>1</td>
</tr>
<tr>
<td>J: Wood frames buildings walls sheathed or plastered.</td>
<td>1 or 2</td>
</tr>
<tr>
<td>K: Wood frame buildings walls without sheathing or plaster</td>
<td>4</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Symmetry (of resisting elements)</th>
<th>Quantity (of resisting elements)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1: Symmetrical</td>
<td>1 Many resisting elements</td>
</tr>
<tr>
<td>2 or 3: Symmetry poor</td>
<td>2 Medium amount of resisting elements.</td>
</tr>
<tr>
<td>3 or 4 : Very unsymmetrical</td>
<td>3 Few resisting elements</td>
</tr>
<tr>
<td>Note: Add 1(not exceed 4) to each rating if a high degree of vertical non-uniformity in stiffness occurs.</td>
<td>4 Very few resisting elements.</td>
</tr>
</tbody>
</table>

**Note:**
If exterior shear walls are at least 75% of building length this rating will be 1.

<table>
<thead>
<tr>
<th>Present condition (of resisting elements)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1: No cracks no damage</td>
</tr>
<tr>
<td>2: Few minor cracks</td>
</tr>
<tr>
<td>3: Many minor cracks or damage</td>
</tr>
<tr>
<td>4: Major cracks or damage</td>
</tr>
</tbody>
</table>

**Note:**
If masonry walls, note quality of mortar – good or poor.
If Lime mortar is poor, use next higher rating.

**Notes of: Horizontal – Resisting E:**

<table>
<thead>
<tr>
<th>Type</th>
<th>Rigidity - Ratings</th>
</tr>
</thead>
<tbody>
<tr>
<td>A: Diaphragm</td>
<td>1.0 Rigid</td>
</tr>
<tr>
<td>B: Steel horizontal bracing</td>
<td>1.5 Semi-rigid</td>
</tr>
<tr>
<td></td>
<td>2.0 Semi-Flexible</td>
</tr>
<tr>
<td></td>
<td>1.5 Flexible</td>
</tr>
</tbody>
</table>
### Intensity level factor:

<table>
<thead>
<tr>
<th>Modified mercalli scale</th>
<th>Intensity level factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>VIII or Greater</td>
<td>1</td>
</tr>
<tr>
<td>VII</td>
<td>2</td>
</tr>
<tr>
<td>VI</td>
<td>3</td>
</tr>
<tr>
<td>V or less</td>
<td>4</td>
</tr>
</tbody>
</table>

Basic structural rating = \( GR + \frac{2}{3}(\text{largest of } SR_1 \text{ and } SR_2) \)

Capacity ratio = \( \frac{\text{Basic Structural Rating}}{\text{Intensity Level Factor}} \)

### 2.4.5 Israeli evaluation method

This method is developed by A.S. Scalat,[15] which aims at providing statistical information regarding the seismic vulnerability of a large group of buildings. This method enables quick but approximate evaluation of buildings in Israel. The building is classified insufficient seismic resistance according to structural score \( S \) which includes \( (S_o) \) basic score which depends on type of structure and seismic zone factor \( Z \) (which defined in seismic map of the country), and \( \Delta S \) modifiers, which are identical for all types of structures and all seismic zones. The modifiers depend on condition of structure and superstructure. The structural score \( S \) is calculated from equation(2.9):

\[
S = S_o + \Delta S
\]  

(2.9)

If \( S < 1 \) denotes insufficient seismic resistance.

\( Z = \) peak ground acceleration / acceleration of gravity.

Where \( Z: \)

<table>
<thead>
<tr>
<th>Zone factor ( (Z) )</th>
<th>Risk Classifications</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25 – 0.3</td>
<td>H = High risk</td>
</tr>
<tr>
<td>0.15 – 0.2</td>
<td>M = Moderate risk</td>
</tr>
<tr>
<td>&lt;0.1</td>
<td>L = Low moderate</td>
</tr>
</tbody>
</table>

#### Table (2.4) Grading system of Israeli method

<table>
<thead>
<tr>
<th>Type of structure S0:</th>
<th>Risk category</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>H</td>
</tr>
<tr>
<td>Wood Frames</td>
<td>2.2</td>
</tr>
<tr>
<td>Steel moment-resisting frames</td>
<td>2</td>
</tr>
<tr>
<td>Braced steel frames</td>
<td>1.5</td>
</tr>
<tr>
<td>Concrete shear walls</td>
<td>2</td>
</tr>
<tr>
<td>Precast concrete large panels</td>
<td>1.5</td>
</tr>
<tr>
<td>Concrete frames</td>
<td>1</td>
</tr>
<tr>
<td>Recast concrete frames</td>
<td>0.5</td>
</tr>
<tr>
<td>Reinforced masonry</td>
<td>1.5</td>
</tr>
<tr>
<td>Infilled frames</td>
<td>0.7</td>
</tr>
<tr>
<td>Plain brick / stone masonry</td>
<td>0.3</td>
</tr>
</tbody>
</table>
Note:
Where several types of structure are present, the predominant type will be considered. When in doubt, the minimum basic score will be chosen.
The Proposed modifiers are identical for all types of structures and all seismic zones:

### Table 2.5 Modifiers ΔS

<table>
<thead>
<tr>
<th>Types of structures</th>
<th>Modifiers ΔS</th>
</tr>
</thead>
<tbody>
<tr>
<td>High-rise buildings (8 stories or more)</td>
<td>-0.5</td>
</tr>
<tr>
<td>Medium-rise buildings (4-7 stories)</td>
<td>0</td>
</tr>
<tr>
<td>Low – rise buildings (3 stories or less)</td>
<td>+0.3</td>
</tr>
<tr>
<td>Poor condition</td>
<td>-0.3</td>
</tr>
<tr>
<td>Poor condition of recast concrete structures</td>
<td>-0.5</td>
</tr>
<tr>
<td>Soft storey</td>
<td>-1</td>
</tr>
<tr>
<td>Significant eccentricity</td>
<td>-0.5</td>
</tr>
<tr>
<td>Pounding possible (for medium &amp; high rise building)</td>
<td>-0.2</td>
</tr>
<tr>
<td>Adjacent slab at same level</td>
<td></td>
</tr>
<tr>
<td>Pounding possible (for medium &amp; high rise building)</td>
<td>-0.5</td>
</tr>
<tr>
<td>Adjacent slab at different levels</td>
<td></td>
</tr>
<tr>
<td>Heavy cladding (precast concrete or cut stone)</td>
<td>-0.5</td>
</tr>
<tr>
<td>Short concrete columns</td>
<td>-0.5</td>
</tr>
<tr>
<td>Year of construction ; Before 1960</td>
<td>-0.5</td>
</tr>
<tr>
<td>1960 – 1975</td>
<td>0</td>
</tr>
<tr>
<td>After 1975</td>
<td>+0.5</td>
</tr>
<tr>
<td>Type of soil</td>
<td>0</td>
</tr>
<tr>
<td>S1 (Rock and stiff clay)</td>
<td>0</td>
</tr>
<tr>
<td>S2 (Stand, gravel)</td>
<td>-0.2</td>
</tr>
<tr>
<td>S3 (Soft and medium soil) or unknown</td>
<td>-0.3</td>
</tr>
<tr>
<td>S3 + (high – rise building)</td>
<td>-0.4</td>
</tr>
</tbody>
</table>

### 2.4.6 Yugoslavian method for field evaluation of existing building [16]

Peter Sheppard and Mariana Lutman developed this method of evaluation. This method was applied on a group of concrete infill buildings, in the old part of city of Ljubljana in Yugoslavia.

In the case of mixture of masonry walls and reinforced concrete columns, it was decided to take into account only five main parameters of seismic vulnerability, with the following weighting factors w :-

1- Type and quality of walls and columns, \( w=1.0 \).
2- Quantity of walls and columns (relative to the floor plan), \( w=1.0 \).
3- Layout of walls and columns in the floor plan, \( w=1.0 \).
4- Mix of the structure and structural details, \( w=1.50 \).
5- Other factors (condition of building etc.) \( w=0.5 \).
In each case an individual parameter is given an integral value ranging from 1 to 5, where the value of 1 means condition in accordance with valued technical regulations and a value of 5 means conditions do not correspond at all to the regulations. The sum of these values, each multiplied by the given weighting factor, is called the basic seismic vulnerability (Ps) of the building, and it can assume values between 5 and 25.

In the case of methodology presented here, however, building height is taken into account by means of an additional parameters (Pn) which assumes the following values:

1. Buildings up to three stories high (ground plus two) of normal story height, $Pn = 0$
2. Buildings three stories high with above average story height, $Pn = 2$
3. Building four stories height with above average story height, and buildings five story height, $Pn = 4$,
4. Buildings five story height, and buildings six stories high with normal story height, $Pn = 6$

Buildings with above average story height are considered as having an average story height exceeds 3.5m.

From the local intensity scale in Slovenia which depends on zone location, then we can get additional parameter ($PI$) Table (2.6).

<table>
<thead>
<tr>
<th>Zone</th>
<th>Seismic coefficient $Ks$</th>
<th>$PI$</th>
</tr>
</thead>
<tbody>
<tr>
<td>VIII - 1</td>
<td>0.04</td>
<td>-3</td>
</tr>
<tr>
<td>VIII - 2</td>
<td>0.05</td>
<td>0</td>
</tr>
<tr>
<td>VIII - 3</td>
<td>0.06</td>
<td>+3</td>
</tr>
<tr>
<td>IX - 1</td>
<td>0.08</td>
<td>+9</td>
</tr>
<tr>
<td>IX - 2</td>
<td>0.10</td>
<td>+15</td>
</tr>
<tr>
<td>IX - 3</td>
<td>0.12</td>
<td>+21</td>
</tr>
</tbody>
</table>

When all parameters mentioned above is taken into account, an expression is written defining seismic vulnerability of older buildings:

$$ V_s = (P_s + P_n + P_i) F_t $$  \hspace{1cm} (2.10)

Where $F_t$ is a factor for the type of load – bearing system, whose value is taken to be 1 for masonry buildings, and 0.85 for buildings with masonry walls and reinforced concrete columns. Then the seismic risk is estimated, which are of particular importance when deciding the priority of individual buildings, the two most important parameters are:

a- The number of the building users.
b- The total usable floor area of building.

The average density of users of the building can be determined from those two parameters, the number of users will change during the year, but for simple analysis it is sufficient to use an average density which is as realistic as possible. First the density of users in individual building is determined (e.g. the number of users per $100m^2$ of usable floor space) and then this figure is compared with the weighted
average density. Density of users (Dr) within the group of buildings can be determined. The relative seismic risk, which can be allocated to an individual building, is then given by the expression: Eq. (2.11)

\[ V_u = V_s D_r \] (2.11)

The relative seismic risk for this method is as follows:

- If \( Vu \geq 25.0 \) The relative seismic risk is very high.
- If \( 17.5 \leq Vu < 25.0 \) The relative seismic risk is high
- If \( 10 \leq Vu < 17.5 \) The relative seismic risk is medium.
- If \( Vu < 10 \) The relative seismic risk is low.

### 2.4.7 Detailed Turkish method [17-20]

The main objective is to identify the buildings that are highly vulnerable to damage, that is the seismic performance is inadequate to survive a strong earthquake. Hence, the damage scores obtained from the derived discriminate functions are used to classify existing buildings as “safe”, “unsafe” and “intermediate”. The discriminate functions are generated based on the basic damage inducing parameters, namely number of stories (\( n \)), minimum normalized lateral stiffness index (\( mnlstfi \)), minimum normalized lateral strength index (\( mnlsi \)), normalized redundancy score (\( nrs \)), soft story index (\( ssi \)) and overhang ratio (\( or \)). In the determination of the estimation variables to be used in the analysis, the basic assumption is that all of the buildings involved in the inventory are exposed to a specific earthquake. In other words, each building stock in itself has faced the same ground motion properties, thus the damage will be evaluated only on the basis of structural responses rather than including the excitation parameters. Considering the characteristics of the damaged structures and the huge size of the existing building stock, the following parameters were chosen as the basic estimation parameters of the proposed method:

1. Number of stories (\( n \)).
2. Minimum normalized lateral stiffness index (\( mnlstfi \)).
3. Minimum normalized lateral strength index (\( mnlsi \)).
4. Normalized redundancy score (\( nrs \)).
5. Soft story index (\( ssi \)).
6. Overhang ratio (\( or \)).

These parameters are briefly defined in the following paragraphs.

**i. Number of stories (\( n \))**: This is the total number of individual floor systems above the ground level.

**ii. Minimum normalized lateral stiffness index (\( mnlstfi \))**: This index is the indication of the lateral rigidity of the ground story, which is usually the most critical story. If the story height, boundary conditions of the individual columns and the properties of the materials used are kept constant, this index would also represent the stiffness of the ground story. This index is calculated by considering the columns and the structural walls at the ground story. While doing this, all vertical reinforced concrete members with “maximum cross-sectional dimension / minimum cross-sectional dimension ratio” less than 7 are considered as columns. All other reinforced concrete structural members are considered as structural walls. The (\( mnlstfi \)) parameter shall be computed based on the following relationship:
\[ mnlstfi = \min \left( I_{nx}, I_{ny} \right) \]  

(2.12)

\( I_{nx} \) and \( I_{ny} \) values are to be calculated by using Eq.(2.13)

\[
I_{ny} = \frac{\sum (I_{col})_y + \sum (I_{sw})_y}{\sum A_f} \times 1000
\]

(2.13)

\[
I_{ny} = \frac{\sum (I_{col})_y + \sum (I_{sw})_y}{\sum A_f} \times 1000
\]

where:

\( \sum (I_{col})_x \) and \( \sum (I_{col})_y \) : Summation of the moment of inertias of columns about their centroidal \( x \) and \( y \) axes, respectively.

\( \sum (I_{sw})_x \) and \( \sum (I_{sw})_y \) : Summation of the moment of inertias of all structural walls about their centroidal \( x \) and \( y \) axes, respectively.

\( I_{nx} \) and \( I_{ny} \) : Total normalized moment of inertia of all members about \( x \) and \( y \) axes, respectively.

\( \sum A_f \) : Total story area above ground level.

iii. Minimum normalized lateral strength index (\( mnlsi \)):

The minimum normalized lateral strength index is the indication of the base shear capacity of the critical story. In the calculation of this index, in addition to the existing columns and structural walls, the presences of unreinforced masonry filler walls are also considered. While doing this, unreinforced masonry filler walls are assumed to carry 10 percent of the shear force that can be carried by a structural wall having the same cross-sectional area [8, 11, and 12]. As in \( mnlstfi \) calculation, the vertical reinforced members with a cross-sectional aspect ratio of 7 or more are classified as structural walls. The \( mnlsi \) parameter shall be calculated by using the following equation:

\[ mnlsi = \min \left( A_{nx}, A_{ny} \right) \]  

(2.14)

where:

\[
A_{nx} = \frac{\sum (A_{col})_x + \sum (A_{sw})_x + .01\sum (A_{sw})_x}{\sum A_f} \times 1000
\]

(2.15)

\[
A_{ny} = \frac{\sum (A_{col})_y + \sum (A_{sw})_y + .01\sum (A_{sw})_y}{\sum A_f} \times 1000
\]

For each column with a cross-sectional area denoted by \( A_{col} \):

\[
(A_{col})_x = K_x \times A_{col}
\]

\[
(A_{col})_y = K_y \times A_{col}
\]  

(2.16)
where:

- \( K_x = \frac{1}{2} \) for square and circular columns
- \( K_x = \frac{2}{3} \) for rectangular columns with \( b_x > b_y \)
- \( K_x = \frac{1}{3} \) for rectangular columns with \( b_x < b_y \); and
- \( K_y = 1-K_x \)

For each shear wall with cross-sectional area denoted by \( A_{sw} \):

\[
\begin{align*}
(A_{sw})_x &= K_x \times A_{sw} \\
(A_{sw})_y &= K_y \times A_{sw}
\end{align*}
\]  

(2.17)

Where,

- \( K_x = 1 \) for structural walls in the direction of x-axis;
- \( K_y = 0 \) for structural walls in the direction of y-axis; and
- \( K_y = 1-K_x \).

For each unreinforced masonry filler wall with no window or door opening and having a cross sectional area denoted by \( A_{mw} \):

\[
\begin{align*}
(A_{mw})_x &= K_x \times A_{mw} \\
(A_{mw})_y &= K_y \times A_{mw}
\end{align*}
\]  

(2.18)

Where,

- \( K_x = 1.0 \) for masonry walls in the direction of x-axis;
- \( K_y = 0 \) for masonry walls in the direction of y-axis; and
- \( K_y = 1-K_x \).

iv. Normalized redundancy score \((nrs)\): Redundancy is the indication of the degree of the continuity of multiple frame lines to distribute lateral forces throughout the structural system.

The normalized redundancy ratio \((NRR)\) of a frame structure is calculated by using the following expression:

\[
NRR = \frac{A_n \left( n_{f_x} - 1 \right) \left( n_{f_y} - 1 \right)}{\sum A_{gf}} \times 1000
\]  

(2.19)

Where:

- \( A_n \): the tributary area for a typical column.
- \( A_n \) shall be taken as 25 m\(^2\) if \( n_{f_x} \) & \( n_{f_y} \) are both greater than and equal to 3
- In all other cases, \( A_n \) shall be taken as 12.5 m\(^2\)
- \( A_{gf} \): the area of the ground story, i.e. the footprint area of the building.
  depending on the value of \( nrr \) computed from Eq. (2.19), the following discrete values are assigned to the normalized redundancy.

Score \((nrs)\):
v. **Soft story index (ssi):** On the ground story, there are usually fewer partition walls than in the upper stories. This situation is one of the main reasons for soft story formations. Since the effects of masonry walls are included in the calculation of \( mnlsi \). Soft story index is defined as the ratio of the height of first story (i.e. the ground story), \( H_1 \), to the height of the second story, \( H_2 \)

\[
ssi = \frac{H_1}{H_2}
\]  

(2.20)

vi. **Overhang ratio (or):** In a typical floor plan, the area beyond the outermost frame lines on all sides are defined as the overhang area. The summation of the overhang area of each story, \( A_{\text{overhang}} \), divided by the area of the ground story, \( A_{gf} \), is defined as the overhang ratio.

\[
or = \frac{A_{\text{overhang}}}{A_{gf}}
\]  

(2.21)

### 2.4.7.1 Statistical Model

The effects of different parameters on seismic damage are vary. In order to make a more rational and systematic evaluation of damage inducing parameters in the prediction of seismic vulnerability of structures, a statistical technique, known as discriminate analysis is adopted. It is possible to evaluate structures at different performance levels according to different objectives. If the main concern is to identify the buildings that are severely damaged or collapsed, the first three damage states (i.e. \( NL \) and \( M \)) can be considered as one group and the severely damaged state and collapsed cases as the other group, reducing the distinct damage states into two. Since the main objective is the identification of severely damaged and collapsed buildings for life safety purposes, this classification can be referred as “Life safety performance classification” (LSPC). Similarly, if the main concern is to identify the structures which suffer no damage or light damage during an earthquake, the first two damage states (\( N \) and \( L \)) can be considered as one group and remaining damage states (\( M, S \) and \( c \)) as the other group, reducing the distinct damage states into two. This identification is named as “Immediate Occupancy Performance Classification” (IOPC) since the main concern is to identify the buildings that can be occupied immediately after a strong ground motion. In the discriminate analysis method, first the set of estimation variables that provides the best discrimination among the groups is identified. These variables are known as the “discriminator variables”. Then a “discriminate function”, which is a linear combination of the discriminator variables, is derived. The values resulting from the discriminate function are known as “discriminate scores”. The final objective of discriminate analysis is to classify future observations into one of the specified groups, based on the values of their discriminate scores.
The nonstandard estimate of discriminate function based on six damage inducing parameters is obtained for life safety performance classification by utilizing the SPSS [13] software and the database constituted after 1999 Düzce earthquake. Here, \( DI_{LS} \) denotes the damage index or the damage score corresponding to the \( LSPC \) and the other parameters are as described. The function given in Eq. (2.22) is referred to as the nonstandard discriminate function, because the nonstandard data are used for computing this discriminate function

\[
DI_{LS} = 0.808n - 0.334mnlsf + 0.107mnlsi - 0.687nrs + 0.508ssi + 3.884or - 2.868
\]

In the case of immediate occupancy performance classification, the nonstandard discriminate function, where \( DI_{IO} \) is the damage score corresponding to \( IOPC \), based on these variables is:

\[
DI_{IO} = 0.808n - 0.334mnlsf + 0.107mnlsi - 0.687nrs + 0.508ssi + 3.884or - 2.868
\]

2.4.7.2 Classification methodology

In this classification methodology, concrete buildings are evaluated according to both performance levels, by using Eqs. (2.22), (2.23) and the final decisions for the damage state of the buildings are achieved by considering the results of the two performance levels simultaneously. Moreover, the number of stories is the most significant variable in both performances classifications. In order to improve the discriminating contribution of other parameters, new cutoff values are selected depending on the number of stories. For this purpose, a functional relationship is derived between the cutoff values and the number of stories, \( n \), by fitting a least squares curve to the available damage data. In the determination of the cutoff function, two constraints are also imposed at each story level. These constraints are:

a. The correct classification rate is required to be at least 70%.

b. The maximum classification error related to damage states leading to life loss i.e. severe damage and collapse) is restricted to be 5%.

The resulting cutoff functions based on number of stories, corresponding to the two types of classification, are as follows:

\[
CF (lspc) = -0.090n^3 + 1.498n^2 - 7.518n + 11.885
\]

\[
CF (iopc) = -0.085n^3 + 1.416n^2 - 6.951n + 9.979
\]

By comparing the \( CF \) values with associated \( DI \) Value calculate performance grouping of the building for life safety performance classification \( (LSPC) \) and immediate occupancy performance classification \( (IOPC) \) as follows:-

\[
\begin{align*}
\text{IF} \; DI_{LS} > CF_{LS} & \; \text{Take} \; PG_{LS} = 1 \\
\text{IF} \; DI_{LS} < CF_{LS} & \; \text{Take} \; PG_{LS} = 0 \\
\text{IF} \; DI_{IO} > CF_{IO} & \; \text{Take} \; PG_{IO} = 1 \\
\text{IF} \; DI_{IO} < CF_{IO} & \; \text{Take} \; PG_{IO} = 0
\end{align*}
\]
Table 2.7. Relationships among different classification criteria.

<table>
<thead>
<tr>
<th>Classification</th>
<th>Indicator Variable in LSPC</th>
<th>Indicator Variable in IOPC</th>
<th>Indicator Variable in Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>SAFE (None or Light Damage)</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>UNSAFE (Severe Damage or Collapse)</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>INTERMEDIATE 1 0 2</td>
<td>1</td>
<td>0</td>
<td>2</td>
</tr>
<tr>
<td>INTERMEDIATE 0 1</td>
<td>0</td>
<td>1</td>
<td>2</td>
</tr>
</tbody>
</table>

As observed in Table (2.7), if the indicator variable is consistently “0” or “1” for both LSPC and IOPC cases, the building are rated as “safe” or “unsafe”, respectively. If there is an inconsistency in the classification, in other words if one gives “0” and the other “1” or vice versa, then no final rating is done and the final decision on the seismic safety of the building is left for a more comprehensive detailed seismic evaluation. As the readers may note, in Table (2.7) all possible ratings are considered, among which the one given in the last row, with an IOPC indicator variable of 0 and LSPC indicator variable of 1 does not have any physical meaning whatsoever. It should be kept in mind that the adopted methodology is a statistical tool and such cases are therefore classified as the cases requiring further study.

2.4.7.3 Extension of detailed Turkish method to other regions [21-23]

The procedure is based on the Duzce damage and therefore is valid for regions that have similar site and source characteristics to that of Duzce. These constraints lead to limited use of the procedure in other regions of the world having different seismicity and site characteristics for this reason, modifications were introduced to the original procedure to allow for its use in other regions of the world. The purpose of the improvement is to capture the relative variation in the ground motion intensity with the magnitude distance to source and the soil type. The spectral displacement ($S_d$) value was selected as the damage inducing ground motion parameter, as it is widely used parameter for expressing the vulnerability of buildings. The general trend of damage curve suggests that the variation of damage with $S_d$ follows the form of exponential function. This inference is used to link the change in $S_d$ to the change to be imposed on the cut off values obtained for Duzce. The proposed procedure is developed on the basis of several assumptions, which are listed below:-

- Attenuation relations are believed to represent the variation of the ground motion adequately.
- Construction practice does not show regional variations.
- Damage pattern observed in the reference site would be the same for other sites that have same distance to source and soil type.
- The assessed building performance does not include the effect of soil induced damage such as liquefaction, foundation settlement and land slide.

The steps involved in this procedure can be outlined as follows:-

Step 1:
Obtain site specific response spectra using an appropriate attenuation model.

Step 2:
Calculate spectral displacement at the fundamental periods of interest.

Step 3:
Plot spectral displacement/n as a function of the fundamental periods (or n), n representing number of stories considered in the Duzce study.

**Step 4:**
Convert spectral displacement to damage index (cut off value) by assuming an exponential relation.

**Step 5:**
Normalize all damage indexes at different sites and distances with the damage index obtained for the reference site.

**Step 6:**
Modify Duzce cut off values by multiplying with the cut off modification coefficients.

Since the objective is to obtain cut off modification coefficients (CMC) to be applied on cut off values (CF). The CMC values are presented in Tables (2.8 – 2.11) for the magnitudes of 5, 6.5, 7.0, and 7.4.

Close inspection of these tables reveal that the CMC value for reference site is (2.25) because of the normalization with respect to this site. Obviously, at better site condition and farther distances cut off values should be larger. These CMC values were multiplied with the respective reference cut off values to obtain the cut off values for other site classes. Modified cut off values are computed merely from Equation (2.25), which can handle negative as well as positive values of reference cut off values.

\[
CF_{\text{modified}} = CF + ABC \times (CF \times (CMC - 1))
\] (2.25)

The cut off modification coefficients for other discrete magnitude values has been obtained using the procedure described above. It is obvious from that there is a clear trend between the site to source distances, shear wave velocity, magnitude and the CMC values. Variation of CMC values with the distances and magnitude. The effects of both distances and magnitude have been incorporated through regression analyses that yielded the relationship given in equation (2.26). The coefficients \(c\) have been obtained from regression of data obtained for each site class (Vs and d) and magnitude (MW) value. The coefficients of the equation are presented in Table (2.12). This equation can be used to obtain the cut off modification for a given site under a given earthquake. The modified cut off values is then used to assess the seismic performance of a given building population by applying the procedure described above.

\[
CMC = C_1^{(1 - C_2 M_0 + C_3 + C_4)}
\] (2.26)

<table>
<thead>
<tr>
<th>V_s (m/s)</th>
<th>Distance</th>
<th>0-4</th>
<th>5-8</th>
<th>9-15</th>
<th>16-25</th>
<th>&gt; 26</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-200</td>
<td></td>
<td>2.46</td>
<td>3.09</td>
<td>4.59</td>
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<tr>
<td>201-400</td>
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<td>4.94</td>
<td>8.79</td>
<td>9.001</td>
</tr>
<tr>
<td>401-700</td>
<td></td>
<td>4.67</td>
<td>5.35</td>
<td>6.79</td>
<td>9.54</td>
<td>9.87</td>
</tr>
<tr>
<td>&gt; 701</td>
<td></td>
<td>5.89</td>
<td>6.76</td>
<td>8.63</td>
<td>12.232</td>
<td>12.66</td>
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### Table (2.9) cut off modification Coefficients (CMC) for Mw = 6.5

<table>
<thead>
<tr>
<th>V&lt;sub&gt;s&lt;/sub&gt; (m/s)</th>
<th>Distance (Km)</th>
<th>0-4</th>
<th>5-8</th>
<th>9-15</th>
<th>16-25</th>
<th>&gt; 26</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-200</td>
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<td>0.871</td>
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<td>201-400</td>
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<td>1.115</td>
<td>1.411</td>
<td>1.809</td>
<td>2.577</td>
<td>4.033</td>
</tr>
<tr>
<td>401-700</td>
<td></td>
<td>1.349</td>
<td>1.727</td>
<td>2.396</td>
<td>3.461</td>
<td>5.471</td>
</tr>
<tr>
<td>&gt;700</td>
<td></td>
<td>1.577</td>
<td>2.116</td>
<td>2.964</td>
<td>4.309</td>
<td>6.832</td>
</tr>
</tbody>
</table>

### Table (2.10) cut off modification Coefficients (CMC) for Mw = 7.0

<table>
<thead>
<tr>
<th>V&lt;sub&gt;s&lt;/sub&gt; (m/s)</th>
<th>Distance (Km)</th>
<th>0-4</th>
<th>5-8</th>
<th>9-15</th>
<th>16-25</th>
<th>&gt; 26</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-200</td>
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<td>0.802</td>
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<td>1.809</td>
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</table>

### Table (2.11) cut off modification coefficients (CMC) for Mw = 7.4

<table>
<thead>
<tr>
<th>V&lt;sub&gt;s&lt;/sub&gt; m/s</th>
<th>Distance (Km)</th>
<th>0-4</th>
<th>5-8</th>
<th>9-15</th>
<th>16-25</th>
<th>&gt; 26</th>
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<tr>
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<td></td>
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<td>1.18</td>
<td>1.53</td>
<td>2.099</td>
<td>3.177</td>
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<tr>
<td>&gt;700</td>
<td></td>
<td>1.052</td>
<td>1.36</td>
<td>1.810</td>
<td>2.534</td>
<td>3.90</td>
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</table>

### Table (2.12) Coefficients of equation (2) for each soil type

<table>
<thead>
<tr>
<th>V&lt;sub&gt;s&lt;/sub&gt; m/s</th>
<th>C1</th>
<th>C2</th>
<th>C3</th>
<th>C4</th>
</tr>
</thead>
<tbody>
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<td>&lt;200</td>
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<td>5.4712</td>
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</tr>
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<td>&gt;700</td>
<td>9.63</td>
<td>0.6572</td>
<td>0.0348</td>
<td>2.655</td>
</tr>
</tbody>
</table>

### 2.5 Summary

According to the detailed study of the previous evaluation methods it is clear that some of these methods can be applied easily to Gaza Strip buildings after some modifications.

1. The preliminary Turkish method is a good quick calculation one, but it can not be used for all types of Gaza Strip buildings, because it is recommended for only low to mid rise reinforced concrete frame building, while in Gaza there are other types of buildings.

2. The second Canadian method is a simple method but it could not be used in Gaza Strip because the factors which are used in the method are depending on the Canadian National Code while there is a big difference between geology of Canada and Palestine geology.

3. Long beach evaluation method is a comprehensive method, it evaluates the structural system, special hazard and physical conditions. It evaluates the buildings into three grades, low hazard, intermediate and serious life hazard. There are some difficulties on applying this method on Gaza like, all structural
system studied and graded for this method are different from the ones used on Gaza Strip. To evaluate any structure system here, you must guess the factors, and this method is found for California which has a seismic zone and geological nature different from Palestine.

4. National Bureau of standards method can be applied to braced and unbraced steel frame, concrete frames, shear walls structures, bearing wall structures and long – span roof structures. In this method a general rating (GR) is evaluated which based on type of structure, the symmetry rating (S) is based on the eccentricity center of mass, quantity rating (Q) which is based on number of vertical resisting elements and horizontal resisting elements are evaluated. The difficulties on applying this method on Gaza are that, all these factors are found for California with nature and geological intensity factor is a big difference to Palestine.

5. The Israeli method depends on the type of structural which has (S) scores, and the conditions of structure and super structure which has (S_o) score. It is clear that this method is more suitable of all the preliminary evaluation method for Gaza Strip, because of the similarity in geological conditions, construction systems and construction materials. Also this method doesn't leaves a wide range for self judgment. However, this method has a limitation in terms of classifying the buildings for earthquake resistance, therefore it needs to be modified which has been done in chapter 5.

6. The Yugoslavian method is applied for one type of structure which is a mixture of masonry walls and reinforced concrete columns. This method is limited for this type of structure so it can not be applied in Gaza Strip and also all factors which found in evaluation are found for Yugoslavian, especially the intensity factor.

7. The detailed Turkish method is based on observed damage and signification building attributes would provide more reliable and accurate results for regional assessments. It is statistical method and depend on calculating the real stiffness of columns and structural walls on ground story, and base shear capacity of the critical story. This method can easily be applied in Gaza Strip.
In this chapter the structural systems, building materials and the buildings used in Gaza Strip are discussed briefly. The evaluation methods which discussed in previous chapter will be applied on Gaza buildings in the next chapter. Buildings in Gaza Strip can be classified according to many categories namely:

b. Structural systems.
c. Building use.

In the following subsections a brief discussion of these items are found:

3.1 Building materials

Different construction materials were used in Gaza Strip according to different categories. A . Masonry. b. Reinforced concrete. c. Steel.

3.1.1 Masonry:
Masonry is most important construction material used in Gaza buildings, it is divided into sand stone, concrete blocks and natural stone.

a. Sand Stone

![Sand stone building in KhanYounis](image)

Fig. 3.1 Sand stone building in KhanYounis

Most of the buildings found in old Gaza city were constructed using sand stone as shown in Fig. (3.1). It has been used for low rise buildings, always one story. These buildings are very weak in resisting seismic loading because of the fact that masonry is a brittle material. However the combination of weight, stiffness and weakness against tensile forces make sand stone buildings highly vulnerable to earthquakes[1].

Since sand stone, which can be stressed relatively high in compression, is weak in resisting bending and shear, collapse is often the result.
b. Concrete blocks

Concrete blocks are very stiff and brittle material, so they are recognized as dangerous in resisting earthquakes[3]. They were used as masonry bearing walls for one story buildings and infill walls in reinforced concrete buildings as shown in Fig. (3.2). When unreinforced masonry walls begin to crack, in terms of engineering analysis, it is usually described as having failed, even if collapse does not occur. The internal elastic strength of the wall blocks, and repeated cycles the wall undergoes plastic deformations through movement along the mortar joints or in bending. The most important attribute of soft mortar is that, when mortar strengths are below that of masonry units, and the wall does not crack, it does so along the mortar joints, resulting in greater overall stability.

c. Natural stones

Natural stones are used as cladding for some buildings constructed in Gaza Strip as shown in Fig. (3.3). During earthquakes the cladding and ornaments from outside of buildings may fail if they were not fixed to the walls. Two methods on fixing the stone cladding are used in Gaza, the first is by putting steel mesh on the exterior walls and buildings one to two courses of the stones and casting concrete
between them. Another method is by fixing the stone with the walls by steel wires. The exterior stone cladding should be properly anchor to the exterior walls for in plane out of plane lateral forced.

3.1.2 Reinforced Concrete

![Fig. 3.4 Reinforced concrete building](image)

The physical properties of reinforced steel which is the ductility and high degree of deformation make the reinforced concrete building absorb the bulk of earthquakes energy. Reinforced concrete systems are better than masonry in resisting seismic loading. High rise buildings were constructed with reinforced concrete as shown in Fig (3.4). Reinforced concrete buildings are rigid by design and their rigidity can be improved further by small increases in steel used. The frame work of such beams and columns can be made to resist earthquake vibrations of considerable magnitudes. Almost the multistory buildings are framed, thus the structures find great support. It must be noted here that the RC frames resist of major portions of earthquake induced forced.

3.1.3 Steel structures

![Fig. 3.5 Steel buildings](image)

Steel is used on construction sport complexes (indoor, stadium, clubs, ect) for long spans roofs as shown in Fig (3.5). Space trusses is used as main girders and plan
trusses as secondary, because of the good properties of steel especially ductility, steel buildings are considered the best in resisting earthquakes. Also steel possesses the highest strength to weight ratio compared to any building material used today. It remains one of the strongest most durable economically manufactured materials.

3.2 Structural systems

Three types of structural systems are mainly used on Gaza Strip:
   A - Masonry bearing walls.
   B - Column and diaphragm system.
   C - Concrete Frame.

3.2.1 Masonry bearing walls

These types of structures were used in regions which were built in the past and in some buildings in the refugee camps. The external walls and some interior walls are bearing walls of unreinforced masonry as shown in Fig. (3.6). The usual floor construction consists of reinforced concrete slab supported by unreinforced masonry bearing walls. These buildings can be consider as unreinforced masonry bearing walls and very weak in resisting seismic loading.

3.2.2 Column and diaphragm system
This structural system is the main system which is used widely in most of Gaza Strip buildings as shown in Fig.(3.7). Up to six stories are not constructed and designed for seismic loading. So that this system is considered weak on resisting seismic loading, but the main parts of the structural system which are accounted for resisting the lateral forced are:

1. The columns and diaphragm, based on the fact that it works as frames, when loaded laterally, in spite that they are not designed to act as a frame.

2. The infill walls which are used as partitions are not designed to carry loads and there are no an exact definition of their strength.

Also rise buildings are constructed by this structural system by adding concrete shear walls to resist lateral loads. These buildings are designed for seismic loading. This system is widely used in Gaza Strip and the shear walls always are the stair walls and core lifts walls can be considered as shear walls[4]. On the other hand, there are many faults which affect resistance to seismic events like:

   i. Concentrations of mass.
   ii. Vertical irregularities.
   iii. Pound of adjacent buildings
   iv. Presence of cantilevers.
   v. Concentration of stress due to complex plans.
   vi. Horizontal irregularities.
   vii. Foundation failures.
   viii. Beam column joint failure.

   a. Concentrations of mass

Fig. 3.8 Concentrations of mass

High concentrations of mass on a given level of the building are problematic. This occurs on floors where heavy items are placed, such as equipment, tanks, store rooms, or filing cabinets. Minarets of mosques in Gaza were built as concentrated mass on roofs as shown on Fig (3.8) which is very dangers during earthquakes. The problem is greater, the higher level is located, due to the fact that seismic response accelerations increase upward, increasing seismic forces and possibility of equipment collapsing and causing structural damage. In architectural, it is recommended that
spaces for usually heavy weights be in basements or in buildings isolated from the main structure.

b. Vertical irregularities

When the stiffness and associated strength are abruptly reduced in a story along the height, earthquake induced deformations tend to concentrate at the flexible or weak story. The concentration of damage in a story leads to large deformation in vertical members. The excessive deformation in vertical members often leads to the failure of these members and the collapse of the story. Soft and weak first stories are especially common in multi-story residential buildings in urban areas, where the first story often is used for open space, commercial facilities or garages. Also most special buildings are built leaving the ground floor without interior walls for their gusts as shown in Fig (3.9). The first – story columns during strong earthquake shaking must resist a large base shear, inevitably leading to large story drift concentrated in that story.

![Fig. 3.9 Soft story](image)

Fig. 3.9 Soft story

c. Pounding of adjacent buildings

![Fig. 3.10 Adjacent buildings](image)

Fig. 3.10 Adjacent buildings
Pounding of adjacent buildings could cause structural damage. Proper distance should be maintained between adjacent buildings. As the planning lows are not constructed for the distances between buildings, this problem is widely found in Gaza Strip as shown in Fig. 3.10. In the case of a series of buildings constructed side by side in some locations, the edge buildings are often pushed outward and suffer severe damage while inner buildings are protected from excessive lateral deformation.

d. Presence of cantilever

![Fig. 3.11 Presence of cantilever](image)

Cantilevers are used in Gaza buildings to increase the plan dimensions and to give the building a good sight. But during earthquakes, dynamic response of cantilever differs from the building response, so extra forces will be attributed in the fixation point, special concerns should be considered in design of cantilever by increasing the reinforcement to sustain attributed forces as shown in Fig. 3.11. Another solution is to increase the stiffness of the cantilever by adding stiff beams even they are not countered for carrying loads. In many buildings stiff beams are added for both increasing the stiffness of cantilever and for architectural aspects. The cantilever elements can cause harmful vertical vibration during heavy earthquakes.

e. Concentration of stress due to complex plans

Concentration of stress arises in buildings with complex floor plans, and is very common in hospital and school buildings. A complex plan is defined as that in which the line joining any two sufficiently distant points lies largely outside of the plan. This occurs when wings of significant size are oriented in different directions, for instance in H, U, or L shapes as shown in Fig. 3.12.
In irregularly shaped floor plans, the wings may be likened to a cantilever built into the remaining body of the building, a point that would suffer smaller lateral distortions than in the rest of the wing. Large concentrations of stress appear in such transition areas, frequently producing damage to the nonstructural element, the vertical structure, and even the diaphragms.

f. Horizontal irregularities

The problems mentioned below refer to the plan of the structure in relation to the form and distribution of architectural space. The configuration problems in the plan arise when the floor plans are continuous that is, when they are not made up of discrete units. Some floor plans that at first glance seem complex, but that rely on seismic expansion joints, may not face performance problems from earthquakes as shown in Fig. 3.13.

g. Foundation failures

The failure of foundations is caused by:

a. Liquefaction and loss of bearing.
b. Landslides.
c. Fault rupture.
d. Compaction of soils.
e. Differential settlement.

It is essential that the foundation system moves in unison during an earthquake. When supports consist largely of isolated column footings, it is advisable to add ties in order
to achieve this and to enable the lateral loads to be shared among all the independent footings.

h. Beam- column joint failure

![Beam-column joint](image)

Fig. 3.14  Beam- column joint

When a moment –resisting frame is designed for weak-beam strong –column behavior, the beam –column joint may be heavily stressed after beam yielding and diagonal cracking may be formed in the connection as shown in Fig 3.14.

3.2.3 Concrete frames

![Concrete frames in school building](image)

Fig. 3.15 Concrete frames in school building

This type of structure is used on public buildings (schools, hospital, parking structures ,and mosques) as shown in Fig. 3.15. As these buildings need big distance between columns, the frame system is considered the best for these buildings. The diaphragms are supported by concrete girders and columns. Moment resisting frame structures which are flexible tend to have long natural periods, which often take then away from resonance with the ground motions. Lateral forces are resisted by the frame and the infill masonry walls.
3.3 Building use
Buildings use have a big effect on choosing the suitable structural system like:

a) Commercial buildings and hotels. In those buildings, the column and diaphragms system with shear walls is used.
b) Public buildings (schools, hospitals, government offices, parking structure): These buildings need big distances between columns. Moment resisting frame is the best used for these buildings.
c) Community social – service facilities. The columns and diaphragms system is used in these buildings.
d) Sport complexes (indoor, stadium, clubs, etc) steel or concrete frames are considered the suitable

3.4 Summary
This chapter was a study of the building materials used in the premises of the Gaza Strip, as well as the construction systems used in the construction. It also shows the weaknesses common to these constructions in terms of their resistance to earthquakes and the impact of the uses of buildings in the selection of the preferred system of construction. This study was a prelude to entry to the various valuation methods on the buildings sector which will be in the next chapter.
CHAPTER "4"
Applicability Of Evaluation Methods To Gaza Strip Buildings

Different researchers suggested several methods for evaluating the earthquake safety of existing buildings. Due to lack of clear criteria and specifications for the use of each method, and the fact that those methods depend on engineering sense and judgment, some criteria are assumed in the preliminary evaluation for evaluating the sample correctly. In this chapter the application of all the preliminary evaluation methods to Gaza buildings will be discussed in details, trying to develop a new method suitable for Gaza buildings.

Long Beach method, National Bureau method, Yugoslavian method, the Preliminary Turkish method and the Detailed Turkish method are applied on sample, a building from Gaza Strip which consists of four stories with a soft story on the ground floor. The aim of applying these evaluation methods is to show the advantages and disadvantages of each method and their suitability for applying on Gaza buildings. After this discussion, a new approach is developed suitable for evaluation of Gaza buildings avoiding most of the disadvantages of these methods.

4.1 Sample of four story building in Gaza

Fig. 4.1 Ground floor of sample

The sample building is a building consisting of four stories constructed in the year 1986. The ground floor was soft story.
4.2 Long Beach evaluation method

The grading system is broken down into five categories: Framing system and/or walls (0 to 40 points), diaphragm and/or bracing system (0 to 20 points), partitions (0 to 20). Special hazards such as building layout or soil conditions (0 to 50), and physical condition (0 to 50 points).

The system used was reinforced concrete and masonry bearing walls; it should take grade of (20).
The Bracing system was well anchored reinforced concrete slabs it should take Grade of (0).
The partitions were few unreinforced masonry, good quality mortar, it should take grade of (0).
Soft story on ground floor it should take grade of (35).
There were minor cracks on the plastering so it should take grade of (10).
By collection of all the items grade the total result will be (65).

Classification: The building is classified as intermediate hazard (IDH).

4.3 National Bureau of standards field evaluation method

\[ SR1 = \frac{(SQR + 2PC)}{3} \quad (4.1) \]

The horizontal resisting elements are evaluated using a special form. A rigidity factor (R) is assigned based on the flexibility of the diaphragms.

The basic structural rating = \[ Gr + 2(\text{largest of } SR_1 \text{ and } SR_2) \] \[ \frac{3}{3} \quad (4.2) \]

The building is rated as good if \((CR \text{ less than } 1)\), fair \((CR \text{ between } 1 \text{ and } 1.4)\), poor \((1.5 \text{ to } 2.0)\) or very poor \((\text{over } 2.0)\).

1-General rating(GR)
The structural system used is classified as item \((E)\) which is masonry shear walls unreinforced grade (4).

2-Symmetry of resisting elements(S).
The building is symmetrical and the grade should be (1).

3-Quantity of resisting elements(Q).
There are medium amount of resisting elements, the grade should be (2).

4-Rigidity(R)
The structural system is classified as semi rigid (1.50).

5-The anchorage confirmed but the capacity is not computed(A)
Probably inadequate. Then the grade should be (2).

6-Chord – ratings (C)
The chords are unknown but probably are present so the grade should be (2).

7-Intensity level factor(I).
The region is under low seismically zone so the modified mercalli scale should be (v) or less which take intensity level factor (4).

Classification: The building is classified as good because CR less than (1)
4.4 Yugoslavian method for field evaluation of existing building

This method is easily applied to Gaza building. The seismic risk is calculated using this equation.

\[ V_s = \left( P_s + P_n + P_i \right) F_t \]  \hspace{1cm} (4.3)

Where \( F_t \) is factor for type of load bearing system, whose value is taken to be (1) for masonry buildings and 0.85 for building with masonry walls and reinforced concrete columns.

To find the basic seismic vulnerability (\( P_s \)):
- Quality of walls and columns is good (\( w = 0 \)).
- Quantity of walls and columns is satisfactory (\( w = 1 \)).
- Layout of walls and columns is good (\( w = 0 \)).
- Mix of structure and structural details is good (\( w = 0.5 \)).
- Condition of the building is satisfactory (\( w = 0.5 \)).

\[ P_s = 2 \]

To find additional parameters (\( P_n \)):
- The building is consist of three stories high (\( P_n = 2 \)).

To find local intensity parameter (\( P_i \)):
- Zone location is the lowest from table(2.6) VIII-1 (\( P_i = -3.0 \)).

\[ V_s = \left( P_s + P_n + P_i \right) F_t = 0.85 \]
\[ D_r = 5.0 \]
\[ V_u = V_s D_r \]
\[ V_u = 0.85 \times 5 = 4.25 < 10 \]

Classification: The relative seismic risk is low.

4.5 Israeli method (IM)

This method enables quick and exact evaluation of buildings. The building is classified into sufficient seismic resistance and insufficient seismic resistance according to structural score (\( S \)) which contains of (50) basic score which depends on type of structure and seismic zone factor (\( Z \)) which defined in seismic map of the country and as modifiers, which are identical for all types of structures and all seismic zone. The modifiers depend on condition of structure and superstructure.

The modifiers depend on condition of structural scores is calculated from the following equation:

\[ S = S_0 + \Delta S \]

If \( S < 1 \) denotes insufficient seismic resistance
If \( S > 1 \) sufficient seismic resistance.

1-Type of structure (\( S_0 \))
- The structural system used is in filled frame and it should take grade of (1.5).

2-Zone factor according to seismic maps(Fig 5.5) \( Z = 0.075 \) (low risk)

3-Modifiers
- The building is medium –rise (4-7 stories) the grade should be (0)
- Poor condition the building is in a good case expects some hair cracks, so the grade should be (0).
  a- Soft story
    The building has a soft story the grade should be (-1).
  b- Significant eccentricity
    The building is symmetrical and there is no any eccentricity.
the grade should be (0).

c- **Pounding effects**
The pounding is possible and the adjacent slab is at the same level. The grade should be (-0.2).

d- **Year of construction**
The building is constructed after 1975, so the grade should be (+0.5).

e- **Type of soil**
The soil is soft and medium or unknown, the grade should be (-0.3).

f- **Short concrete columns**
It is referred to any columns between two opening and in our building there are not any short columns so the grade should be (0).

After collecting all the modifies the result will be

\[ \Delta s = -0.8 \]

\[ S = S_o + \Delta s \]

\[ S = 0.7 \]

\[ S<1 \] denotes insufficient seismic resistance

**Classification**: The building is classified as insufficient of seismic resistance.

### 4.6 Preliminary Turkish method

<table>
<thead>
<tr>
<th>Number Of stories</th>
<th>Σ Floor area (m²)</th>
<th>Σ A col (m²)</th>
<th>Σ A col-x (m²)</th>
<th>Σ A col-Y (m²)</th>
<th>Σ A sw-x (m²)</th>
<th>Σ A sw-y (m²)</th>
<th>Σ Amw-x (m²)</th>
<th>Σ Amw-Y (m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>270</td>
<td>3.45</td>
<td>1.45</td>
<td>2</td>
<td>0</td>
<td>0</td>
<td>1.2</td>
<td>1.6</td>
</tr>
</tbody>
</table>

**BCPI = 1.81**

The capacity index is larger than (1.2).

**Classification**: The building is classified as having sufficient capacity against earthquakes and was identified as adequate.

### 4.7 Details Turkish method

Applying this method on the case study building will give the following results. The damage index or damage score corresponding to the life safety performance classification \((DI_{LS})\) shall be computed from this function:

\[ DI_{LS} = .62n -.246mnlstfi -.182mnlisi -.699nrs + 3.269ssi + 2.728or - 4.905 \]

In the case of immediate occupancy performance classification \((IOPC)\), the discriminate function where \(DI_{IO}\) is damage score based on these variables is:-

\[ DI_{IO} = .808n -.334mnlstfi -.107mnlisi -.687nrs + .508ssi + 3.884or - 2.868 \]

<table>
<thead>
<tr>
<th>Number of stories</th>
<th>Σ floors area (m²)</th>
<th>Σ A col m²</th>
<th>Σ A Sw</th>
<th>Σ Amw</th>
<th>I col (x)m4</th>
<th>I col (y)m4</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>270</td>
<td>3.3</td>
<td>0</td>
<td>2</td>
<td>0.0655</td>
<td>0.0506</td>
</tr>
</tbody>
</table>
Minimum normalized lateral stiffness index (monistic)

\[ mnlstfi = \min \left( I_{nx}, I_{ny} \right) = 0.187 \]

\[ mnls = I \min \left( A_{nx}, A_{ny} \right) = 4.8 \]

Normalized redundancy score (NRS):

\[ nrr = \frac{A_{n} \left( nf_{x} - 1 \right) \left( nf_{y} - 1 \right)}{\sum A_{gf}} \times 1000 \]

\[ nrs = 1.0 \]

Soft Story index (ssi)

\[ SSI = \frac{H_{1}}{H_{2}} = 1.33 \]

Overhang ratio (or)

\[ or = \frac{A_{overhang}}{A_{gf}} = 0.1185 \]

\[ DI_{LS} = 0.007436 \]

\[ DI_{LO} = 0.1158 \]

\[ CF_{LS} = 0.395 \]

\[ CF_{IO} = -0.422 \]

\[ DI_{LS} < CF_{LS}. \]

\[ DI_{IO} > CF_{IO} \]

**Classification**: The building is classified as intermediate degree of hazard.
Table 4.1 Results of application of the evaluation methods on the sample

<table>
<thead>
<tr>
<th>Evaluation method</th>
<th>Buildings type</th>
<th>Classification results</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Safe</td>
</tr>
<tr>
<td>Long beach method</td>
<td>Four stories with G.floor soft story</td>
<td>✓</td>
</tr>
<tr>
<td>National bureau method</td>
<td></td>
<td>✓</td>
</tr>
<tr>
<td>Yugoslavian method</td>
<td></td>
<td>✓</td>
</tr>
<tr>
<td>Israeli method</td>
<td></td>
<td>✓</td>
</tr>
<tr>
<td>The first Turkish method</td>
<td></td>
<td>✓</td>
</tr>
<tr>
<td>The second Turkish method</td>
<td></td>
<td>✓</td>
</tr>
</tbody>
</table>

4.8 Analysis of results

It is important to know that the results of long Beach Method and the detailed Turkish statistical method have the same classification criteria for the sample building which is intermediate degree of hazard. Israeli evaluation method classified the building as has insufficient degree of safety. These three methods have logical results because the sample which applied is weak in resisting earthquake, for that of the soft story.

The other methods which are the National Bureau of standard, the preliminary seismic performance assessment procedure (First Turkish method) and Yugoslavian method have the same result which is adequate for resisting earthquakes. The results are illogic because this sample is weak in resisting earthquakes, so these three methods are not preferred to use in Gaza Strip.

The main problem in comparing between long Beach method, the detailed Turkish method and Israeli method was that \( L.B.M \) and detailed Turkish method classifies buildings into three degrees of hazard low, intermediate and serious degree of hazard. But Israeli method \( (IM) \) classifies buildings due to \( E.Q \) resistance only into two degrees of hazard sufficient and insufficient resistance so for making logical comparison and getting clear results the categories for the methods should be the same.

\( L.B.M \) concentrates on special hazard of the building, which includes any non structural elements such as poor quality mortar and non bearing unreinforced masonry walls, also \( L.B.M \) gives a considerable weight for physical condition such as cracks and deterioration of structural elements. These two points in a building are just enough to consider it as seismically insufficient. On the contrast of that those two points are collected in \( (IM) \) under a title for poor condition which has a small grade (-0.3) compared with other factors.

Israeli method takes short columns factor with grade of (-0.50) but LBM dose not consider this factor despite of its importance, it is included in factor of excessive wall
openings taking low grade. With respect to structural system there are two main systems
in Gaza, bearing walls and skeleton type.

On the other hand the second Turkish method is a statistical method which depends
really on the stiffness and the strength of columns, shear walls and filling walls. Also
many factors are involving in evaluation equation, like number of stories and soft story
effects. The classification of this method is realistic, logical and suitable to use on Gaza
strip after the dissection the extension of this method to other regions and comparison
between turkey and Palestine in terms of earthquakes.

By examining results obtained form applying preliminary evaluation methods. It is
noticed that the Israeli method is the most suitable method to apply in Gaza strip due to
the following reasons:

a- Similarity in the geological conditions for Gaza and occupied territories which
appears in choosing zone factor.
b- Construction systems and construction materials are the same. It is easy to apply
relatively with other method and dose not leaves a wide range for self Judgment.
c- It takes into consideration many factors such as building height eccentricity, soft
story, condition of building, pounding, soil factor and short column.
So, based on the previous points, the Israeli method would be more suitable for
preliminary evaluation of Gaza buildings.

However, Israeli method classifies the buildings due to earthquakes resistance only into
two degrees of hazard, sufficient and insufficient resistance. Because preliminary
methods are approximate once and depended on sense of evaluator so minor difference
in scores may transform the building from being sufficient one or in other words it is
not fair or logic to classify all buildings into either sufficient or insufficient. So at least
one intermediate stage should be present also, it is noticed on the Israeli method that all
the buildings evaluation is for the configuration without any study for the lateral
stiffness and lateral strength of the building.

So it is recommended to make a new approach by combining the Israeli method with
detailed Turkish method after the amendment. By this approach the building is
evaluated for its configuration, lateral strength and stiffness. The buildings evaluation
are started by Israeli method to classify the building into sufficient or insufficient
seismic resistance, and if the of results are sufficient, the evaluation process stopped and
the building resistance to seismic loads are enough. If the evaluation results are
insufficient evaluation process continue through the approach. The damage index or the
damage score must found corresponding to the life safety performance.

Classification ($D_{ILS}$) shall be computed from this function.

$$D_{ILS} = 0.62n - 0.246\ln l - 0.182l - 0.699r + 3.269s + 2.728o - 4.905$$

Or in the case of immediate occupancy performance classification ($IOPC$) the
discriminate function where $D_{IO}$ is the damage score.

$$D_{IO} = 0.808n - 0.334\ln l - 0.107l - 0.687r + 0.508s + 3.884o - 2.868$$

The cut of values is confined by applying the cutoff functions corresponding to the two
types of classification as follows:-
\[ CF(lspc) = -0.090n^3 + 1.498n^2 - 7.518n + 11.885 \]
\[ CF(iopc) = -0.085n^3 + 1.416n^2 - 6.951n + 9.979 \]

By comparing the \( CF \) values with \( DI \) values calculate performance grouping of the building for life performance classification (LSPC) and immediate occupancy performance classification (IOPC) as follows:

1. \( DI_{ls} > CF_{ls} \) Take \( PG_{ls} = 1 \)
2. \( DI_{ls} < CF_{ls} \) Take \( PG_{ls} = 0 \)
3. \( DI_{io} > CF_{io} \) Take \( PG_{io} = 1 \)
4. \( DI_{io} < CF_{io} \) Take \( PG_{io} = 0 \)

The indicator variable (0) corresponds to none, light or moderate damage in case of LSPC and none or light damage in the case of IOPC. Similarly the indicator variable (1) corresponds to server damage or collapse in the case of LSPC and moderate or severe damage or collapse in the case of IOPC. In the final stage of the classification procedure, the building is rated as safe (i.e. none or light damage) or unsafe (i.e. severe damage or collapse) or intermediate depending on the values of indicator variables obtained from both classification types.
CHAPTER "5"
Proposed Approach For Evaluations Gaza Strip Buildings To Earthquakes Resistance.

The method is proposed to combine and modify two of existing evaluation methods of. The first is the Israeli method after modifications for a rapid survey of the buildings. If the evaluation is sufficient, the new method will adopt the result. If the evaluation result is insufficient, the evaluation process will continue through the Turkish method for reaching the final outcome of the evaluation at any degree of seismic intensity, where it will classify the buildings into sufficient, intermediate, and insufficient of applying the new method.

1. If the final result is sufficient the building will be accordingly classified as to have enough resistance to earthquakes.
2. If the results of applying the new method is intermediate, the building is classified as in need for strengthening.
3. If the result was insufficient, the building is classified accordingly and the recommendation is to demolish it or it needs major strengthening.

5.1 Comments on Israeli method

Israeli method classifies building due to E.Q resistance into either sufficient or insufficient. In General preliminary evaluation methods are approximate methods and depend on the sense of evaluator. So minor difference in scores may transform the building form being sufficient for seismic resistance to insufficient one or other words it is not fair to classify all the building in either sufficient or insufficient only. So one intermediate stage is suggested. This grade will be the average between them. From comparing the results of other methods during application.

\[
\begin{align*}
\text{If } S < 0.5 & \Rightarrow \text{ insufficient E.Q resistance.} \\
\text{If } 0.5 \leq S \leq 1.0 & \Rightarrow \text{ intermediate E.Q resistance} \\
\text{If } S > 1.0 & \Rightarrow \text{ sufficient E.Q resistance.}
\end{align*}
\]

Israeli method classifies buildings according to construction year into three categories which are before 1960, 1960-1975 and after 1975 and grade is given for each category. Final score of any building is an automatically subtract by a factor (0.5) just because it was built before 1960 even if the whole condition of the building is good or adding grade (0.5) if the building after 1975. No direct relationship between age of the building and seismic resistance, the modification on this item in Gaza so if the building built before 1967 (before the Israel occupation to Gaza Strip) with poor conditions the score subtract (0.5) and if it had a good condition the score subtract (0.2). Else if the building built after 1980 (construction reinforced concrete buildings increases after this date according to municipality records) with poor condition the score add only 0.2 and if it had. For building with a good condition the score added (0.50). Israeli method classify the building according to the soft story which take grade of (-1), which is very large grade. For example for concrete frame building with soft story, the building will classify insufficient seismic resistance although the structural system is good only because the building has soft story.

Also what grade should be given to incomplete soft story, from practice many buildings built as semi soft story, that is have of the ground floor is without walls and the other have is as apartment. Fig. (5.1). So the soft story factor is suggested to rearrange which is in grading from (0) to (-0.5) according to degree of softness of story.
5.2 Modified Israeli evaluation method

From the above discussion the method can be modified to Gaza buildings as follows:

a) \( S = S_0 + \Delta S \)

\( S_0 \) is the basic score from Table (5.1) \( \Delta S \) is modifiers from Table (5.2)

\( Z = \frac{\text{peak ground acceleration}}{\text{acceleration of gravity}} \)

Where \( Z \)

<table>
<thead>
<tr>
<th>( Z )</th>
<th>Risk category</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25-0.30</td>
<td>H = High risk</td>
</tr>
<tr>
<td>0.15-0.20</td>
<td>M = Moderate risk</td>
</tr>
<tr>
<td>&lt; 0.1</td>
<td>L = Low moderate</td>
</tr>
</tbody>
</table>

**Table 5.1 Type of structures (\( S_0 \))**

<table>
<thead>
<tr>
<th>Type of structure</th>
<th>Risk category</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>H</td>
</tr>
<tr>
<td>Wood Frames</td>
<td>2.2</td>
</tr>
<tr>
<td>Steel moment-resisting frames</td>
<td>2</td>
</tr>
<tr>
<td>Braced steel frames</td>
<td>1.5</td>
</tr>
<tr>
<td>Concrete shear walls</td>
<td>2</td>
</tr>
<tr>
<td>Precast concrete large panels</td>
<td>1.5</td>
</tr>
<tr>
<td>Concrete frames</td>
<td>1</td>
</tr>
<tr>
<td>Precast concrete frames</td>
<td>0.5</td>
</tr>
<tr>
<td>Reinforced masonry</td>
<td>1.5</td>
</tr>
<tr>
<td>Infilled frames</td>
<td>0.7</td>
</tr>
<tr>
<td>Plain brick / stone masonry</td>
<td>0.3</td>
</tr>
</tbody>
</table>

From this table we can find \( S_0 \)
### Table 5.2 Modifiers $\Delta S$

<table>
<thead>
<tr>
<th>Types of structures</th>
<th>Modifiers $\Delta S$</th>
</tr>
</thead>
<tbody>
<tr>
<td>High-rise buildings (8 stories or more)</td>
<td>-0.5</td>
</tr>
<tr>
<td>Medium-rise buildings (4-7 stories)</td>
<td>0</td>
</tr>
<tr>
<td>Low-rise buildings (3 stories or less)</td>
<td>+0.3</td>
</tr>
<tr>
<td>Poor condition</td>
<td>-0.3</td>
</tr>
<tr>
<td>Poor condition of precast concrete structures</td>
<td>-0.5</td>
</tr>
<tr>
<td><strong>Soft story</strong></td>
<td></td>
</tr>
<tr>
<td>a- semi soft story</td>
<td>-0.2</td>
</tr>
<tr>
<td>b- complete soft story</td>
<td>-0.5</td>
</tr>
<tr>
<td>Significant eccentricity</td>
<td>-0.5</td>
</tr>
<tr>
<td>Pounding possible (for medium &amp; high rise building) Adjacent slab at same level</td>
<td>-0.2</td>
</tr>
<tr>
<td>Pounding possible (for medium &amp; high rise building) Adjacent slab at different levels</td>
<td>-0.5</td>
</tr>
<tr>
<td>Heavy cladding (precast concrete or cut stone)</td>
<td>-0.5</td>
</tr>
<tr>
<td>Short concrete columns</td>
<td>-0.5</td>
</tr>
<tr>
<td><strong>Year of construction: Before 1967</strong></td>
<td></td>
</tr>
<tr>
<td>a- Poor conditions</td>
<td>-0.5</td>
</tr>
<tr>
<td>b- Good conditions</td>
<td>-0.2</td>
</tr>
<tr>
<td>1967-1980</td>
<td>0</td>
</tr>
<tr>
<td><strong>After 1980</strong></td>
<td></td>
</tr>
<tr>
<td>a- Poor conditions</td>
<td>+0.2</td>
</tr>
<tr>
<td>b- Good conditions</td>
<td>+0.5</td>
</tr>
<tr>
<td><strong>Type of soil</strong></td>
<td></td>
</tr>
<tr>
<td>S1 (Rock and stiff clay)</td>
<td>0</td>
</tr>
<tr>
<td>S2 (sand, gravel)</td>
<td>-0.2</td>
</tr>
<tr>
<td>S3 (soft and medium soil) or unknown</td>
<td>-0.3</td>
</tr>
<tr>
<td>S3 + (High-rise building)</td>
<td>-0.4</td>
</tr>
</tbody>
</table>

From this Table (5.2) we can find $\Delta S$

\[ S = S_o + \Delta S \]

If \( S < 0.5 \) \( \Rightarrow \) Insufficient E.Q resistance.
If \( 0.5 \leq S \leq 1.0 \) \( \Rightarrow \) Intermediate E.Q resistance
If \( S > 1.0 \) \( \Rightarrow \) Sufficient EQ resistance.

### 5.3 Comparison in terms of earthquakes between Turkey and Palestine

#### 5.3.1 Zone factor

Turkey lies within the Mediterranean sector of the alpine – Himalayan system, which extends from Italy to Burma. This system with high mountain ranges and shallow, somewhat diffuse seismicity, constitute one of the most seismically active continental regions of the world with along and well documented history of earthquakes. Portrayal of seismicity and the tectonics of a region provide the essential information towards the assessment of seismic source zones. The epicenter maps of the historical damaging regions as hazardous zones [23].

The tectonics of the region is controlled by the collision of the Arabian and Eurasian Plates as shown in (Figure 5.3). The northward motion of the Arabian plate relative to Eurasia causes lateral escape of the Anatolian block to the west and the northeast
Anatolian block to the east. The Anatolian block is bounded to the north and to the south-east by the north Anatolian and the east Anatolian faults, respectively. The north Anatolian fault (NAF) is the most eminent tectonic feature of the region and one of the best known strike slip faults in the world. It is an about 1500 km long, seismically active right lateral strike-slip fault system extending from the karl ova triple junction in eastern Turkey to mainland Greece. Estimations on the age of the north Anatolian fault range from late Miocene to early Pliocene The total relative displacement along the fault varies from 40 km in the east, near Erzincan, to 15 km in the west, near the Marmora Sea.

The East Anatolian fault zone is a 550 km-long, approximately northeast-trending, left lateral strike-slip fault zone. The fault zone takes up the relative motion between the Anatolian and the Eurasian plates and, between the Arabian and African plates. The east Anatolian fault zone extends from Karloova triple junction in the northeast to the maraş triple junction in the southwest were it intersects the Dead Sea fault. The age of the East Anatolian fault is also highly debated... Only a few major earthquakes occurred on the East Anatolian Fault during the 20th century. Among those, the location of May 22, 1971 Bingol earthquake is very close to the present earthquake which is associated with the east Anatolian fault system. The northeastern segment of the east Anatolian fault between the Karloova triple junction and Bingol is about 60 km long and is composed of many closely spaced parallel strike-slip faults strands. The 1971 Bingol earthquake (Figure 5.3) produced surface breaks mostly along the southwestern half of this segment. The relocated epicenter for the 1971 earthquake is at the southwestern end of the surface breaks. Although the exact locations are not known, two historical, earthquakes of similar size, namely the 1789 and the 1875 events, are reported to have occurred in the vicinity of this segment [24].

The 1971 event, preceded by a few foreshocks, caused considerable damage at Bingöl and its vicinity. Death of 881 and injury of 1157 people have been reported. 3965 housing units were collapsed, 6950 housing units were heavily damaged, 9847 were moderately and 350 housing units were slightly damaged. The earthquake produced a
belt of surface breaks, approximately 38 km long, from Bingöl to Çobantasi. Both field observations on surface breaks and earthquake mechanism solutions reveal left-lateral strike slip geometry [24].

Figure 5.3 Tectonic map of the eastern part of the Anatolian fault zone [25]

Intensity is a subjective measure of cultural damage done by an earthquake. Intensity values are heavily dependent on type of structure, construction method, population density, type of ground and distance from earthquake. The highest PGA recordings were from Bolu (0.822g) and Duzce (0.535g) stations.

Seismic safety in any earthquake is accomplished through an effective building code process which calls for skillful integration by scientists and engineers of factors which balance the elastic demand on the structure with the elastic and inelastic capacity of the structure.

In Palestine, there major tectonic plates face each other and compose the tectonic features that bond the created Palestine-Sinai sandwiched sub plate. The region could be affected by several seismic tectonic sources distributed along the following major tectonic feature of region see (Fig.5.4).

1- The dead sea strike slip fault system that extend to about 1100km from the gulf of Aqaba triple junction up to the collision plate boundary between Arabia and Anatolian sub-plate.
2- The East Anatolian fault that characterized by a left lateral movement continue to meet with the Cyprian Arc.
3- The Carmel Rupture Zone that extend more than 130km of North Western Trend.
4- The Gulf of Suez that follow the same trend of the Mid Red Sea spreading ridge.

Many earthquakes were recorded in many localities in Jordan especially in Dead Sea and Aqaba areas. The major earthquake was recorded by the Acceleograph stations is the main shock of Aqaba earthquake sequence. The maximum ground acceleration is 0.156g and 0.25g.
Fig. 5.4 Tectonic sketch map.

From the equivalent lateral force analysis using IBC (2003).

\[ V = C_S W \]

\( V \) = seismic base shear
\( C_S \) = the seismic response coefficient.

Where:

\[ C_S = \frac{S_{DS}}{R / I_E} \leq \frac{S_{D1}}{(R / I_E)} \]  \hspace{1cm} (5.1)

\( S_{DS} \) = the design spectral acceleration at short period
\( R \) = the response modification factor, it depends on the type of structural system
\( I \) = the occupancy importance factor.

\( S_{DS} = \frac{2}{3} F_a S_s \)
\( S_{D1} = \frac{2}{3} F_v S_l \)

\( S_{D1} \) = the design spectral acceleration at long period
\( F_a \) = short period site coefficient given in table (5.3)
\( F_v \) = long period site coefficient given in table (5.4)
\( S_s \) = spectral response acceleration for short time given in table (5.5)
\( S_l \) = spectral response acceleration for long time given in table (5.5)

As the factors \( R, I_E \) depends on structural system and the importance of the building; they will not affect by location of the structure. Also \( F_a \) has very little difference on the same soil layer. The important factor in the comparison between Turkey and Palestine is \( S_s \).
### Table 5.3 Site coefficient $F_a$

<table>
<thead>
<tr>
<th>Site Class</th>
<th>$S_s &lt; 0.25$</th>
<th>$S_s = 0.5$</th>
<th>$S_s = 0.75$</th>
<th>$S_s = 1.0$</th>
<th>$S_s &gt; 1.25$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.80</td>
<td>0.80</td>
<td>0.80</td>
<td>0.80</td>
<td>0.80</td>
</tr>
<tr>
<td>B</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>C</td>
<td>1.20</td>
<td>1.20</td>
<td>1.10</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>D</td>
<td>1.60</td>
<td>1.40</td>
<td>1.20</td>
<td>1.10</td>
<td>1.0</td>
</tr>
<tr>
<td>E</td>
<td>2.50</td>
<td>1.70</td>
<td>1.20</td>
<td>0.90</td>
<td>0.90</td>
</tr>
</tbody>
</table>

### Table 5.4 Site coefficient $F_v$

<table>
<thead>
<tr>
<th>Site Class</th>
<th>$S_s &lt; 0.10$</th>
<th>$S_s = 0.20$</th>
<th>$S_s = 0.30$</th>
<th>$S_s = 0.40$</th>
<th>$S_s &gt; 0.50$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.80</td>
<td>0.80</td>
<td>0.80</td>
<td>0.80</td>
<td>0.80</td>
</tr>
<tr>
<td>B</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>C</td>
<td>1.70</td>
<td>1.60</td>
<td>1.50</td>
<td>1.40</td>
<td>1.30</td>
</tr>
<tr>
<td>D</td>
<td>2.40</td>
<td>2.0</td>
<td>1.80</td>
<td>1.60</td>
<td>1.50</td>
</tr>
<tr>
<td>E</td>
<td>3.50</td>
<td>3.20</td>
<td>2.80</td>
<td>2.40</td>
<td>2.40</td>
</tr>
</tbody>
</table>

### Table 5.5 (MCE spectral acceleration for different seism city levels.)

<table>
<thead>
<tr>
<th>Region of Seism city</th>
<th>$S_s$</th>
<th>$S_1$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very high</td>
<td>$&gt;1.25g$</td>
<td>$&gt;0.50g$</td>
</tr>
<tr>
<td>High</td>
<td>$&lt;1.25g$, $&gt;0.75g$</td>
<td>$&lt;0.50g$, $&gt;0.30g$</td>
</tr>
<tr>
<td>Moderate to high</td>
<td>$&lt;0.75g$, $&gt;0.35g$</td>
<td>$&lt;0.30g$, $&gt;0.14g$</td>
</tr>
<tr>
<td>Moderate – low</td>
<td>$&lt;0.35g$, $&gt;0.25g$</td>
<td>$&lt;0.14g$, $&gt;0.10g$</td>
</tr>
<tr>
<td>Low</td>
<td>$&lt;0.25g$</td>
<td>$&lt;0.10g$</td>
</tr>
</tbody>
</table>

### Table 5.6 (Site classifications)

<table>
<thead>
<tr>
<th>Class A</th>
<th>Hard rock with measured shear velocity $V_S &gt; 5000$ ft/sec</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class B</td>
<td>Rock with $2500$ ft/sec $V_S &lt; 5000$ ft/sec</td>
</tr>
<tr>
<td>Class C</td>
<td>Very dense soil and soft rock with $1200$ ft/sec $&lt; V_S \leq 2500$ ft/sec, or with either $N &gt; 50$ or $S_u &gt; 2000$ psf</td>
</tr>
<tr>
<td>Class D</td>
<td>Stiff soil with $600$ ft/sec $\leq V_S \leq 1200$ ft/sec, or with either $15 \leq N \leq 50$ or $100$ psf $\leq S_u &lt; 2000$ psf</td>
</tr>
<tr>
<td>Class E</td>
<td>A soil profile $V_S &lt; 6000$ ft/sec, or with either $N &lt; 15$, $S_u &lt; 1000$ psf, or any profile with more than $10$ ft ($3m$) of soft clay defined as Soil with $P_i &gt; 20$ W $\geq 40$ percent</td>
</tr>
</tbody>
</table>

### Comparison between Turkey and Palestine

\[
C_{S(T)} = \frac{2}{3} F_a S_s \frac{R}{I_E} \\
C_{S(P)} = \frac{2}{3} F_a S_s
\]

As Turkey region is classified as high seismic $S_s$ is equal to one and the peak ground acceleration at Bolu equal $0.822g$.
Palestine region is classified as low seismicity. $S_s$ is equal $<0.25$ but from peak ground acceleration from EL Najah seismic Hazard Map (26) . PGA (Dead Sea region) $= 0.35$
PGA (West bank) $= 0.15$
PGA (Gaza strip) $= 0.075$
5.3.2 Soil factor

Turkey can be thought of as a collage of different pieces (possibly terranes) of ancient continental and oceanic lithosphere stuck together by younger igneous, volcanic and sedimentary rocks. Turkey geologically is part of the great alpine belt that extends from the Atlantic Ocean to the Himalayan Mountains. As a result, Turkey is one of the world's more active earthquake and volcano regions. During the Cenozoic folding, faulting and uplifting accompanied by volcanic activity and intrusion of igneous rocks was related to major continental collision between the larger Arabian Eurasian plates [19]. Turkey's varied landscapes are the products of complex earth movements that have shaped Anatolia over thousands of years and still manifest themselves in fair frequent earthquakes and occasional volcanic eruptions. Although, the hazard of strong ground shaking is generally the principal cause of damage to buildings and other structures during earthquakes, these hazards include:

1. Surface fault rupture, which is the direct, shearing displacement occurring along the surface trace of the fault that slips during an earthquake.
2. Soil liquefaction in, which certain types of soil deposits below the ground water table may lose a substantial amount of strength due to strong earthquake ground shaking, potentially resulting in reduced foundation bearing capacity, lateral spreading, settlement and other adverse effects.
3. Land sliding of soil and rock masses on hillside slopes, due to earthquake ground shaking induced inertia forces in the slope.

In Izmir built up area landslides are at two different regions. First of all can be seen in the bed of Kocacay stream. Landslide and rock fall areas are around the metropolitan area.
City especially squatter areas are risky regions about them In Izmir built area, there are 15 different rock fall and landslide areas that are around the city [27]. The geological review of the area classified the geological unit's local site classes as shown on fig.5.6 All the softer superficial deposits, which can increase the effect of earthquake ground shaking. The stiffer soils and rocks were considered to be other zone. Seismically induced soil liquefaction in a phenomenon in which lose to medium dense saturated granular materials develop high pore water pressure and lose shear strength due to cyclic ground vibration included by earthquakes soil liquefaction can also results in instabilities and lateral deformation in areas of sloping ground. Expansive soils are those that contain signification amounts of clays that expand when wetted and can cause damage to foundations if moisture collects beneath structures some potential for fine ground expansive material may be present in the antelope valley. In Altinday – Merkez district, characteristic slit and muddy debris flows were more common and clayey and silt materials are dominate there. An active silt mud flow in central district of Altinday district. Topography and geomorphology can play a large role in the structure. The main topographic and lower karsiyak plain. The plain formed by fine – grained alluvial deposits by water.

![Fig. 5.6 Landslide and rock fall areas of Izmir metropolitan area.][25]

The documented history of Palestine region provided a wealth of descriptions of destructive earthquakes. The Dead Sea rift is transform boundary between the Arabian and African plates, connecting the red sea spreading center in the south to Taurus – Zagros collision zone in the north. The Dead Sea transform is about 1100km. The Jordan valley is a major part of Dead Sea transform, and the most seismically active region in the Middle East. The landscape of the region is dominated by the raft valley a 20-30 km wide valley that is sunk between the eastern high lands of Jordan and Palestinian territories. Much of the
rift valley is below sea level, including the deepest point on the earth surface. The north–south oriented rift valley was also an important migration route for early humans.

The Dead Sea rift is not a true tectonic rift, but a strike–slip fault system or a continental transform that laterally offsets the Arabian tectonic plate against the African tectonic plate. The Dead Sea section exposes beds that experienced soft–sediment deformation. These beds consist of mixture of fine–grained dark clay and silt, with laminated, tabular fragments of aragonite and liquefied coarse sands. Below and above each of these beds, the sequences are laminated and undisturbed, with no preferred orientation or any other indicators of transported sediments. The lateral distribution of the deformed units is not uniform; several of them extend over a large distance and can be traced and correlated among exposures.[28]

The Jordan Valley which comprises one of the lowest depressions of the earth has been formed as a result of an earth fissure and is the most part of it covered by Alluvial marls which frequently display a dissected topography. Tertiary limestone also occurs in some localities. Eastern heights central high lands and the semi coastal region, consists of sedimentation Eocene, turning, and sediments lime stones. Whilst the Cinemania and turning lime stones are mostly very hard and resemble marble, the sedimentation and Eocene lime stones are generally of soft and chalky nature. [29] Gaza Strip region has a substratum of tertiary limes tones, calcareous sand stone marls, clay and deposits cover wide stretches of land. The dune sands are often cemented by calcareous sediments and cemented in filtration and from therefore compact masses of hard rocks see (Fig.5.8).

![Fig.5.7 Soil map of Gaza strip. [26]](image-url)
The Gaza Strip is 360km² has several majors soil type as shown in (Fig.5.8). Sandy losses sand soil kurkar and clay soils are examples of these soils. Sandy soils are moderately calcareous (5-8% C Co₃) loses sandy soils can be found some 5km inland in the central and southern part of the strip is zone along Khan youngish toward Rajah parallel to the coast. This belt forms a transitional zone between the sandy soils and the clay soil. The clay soils are found in the area between the city of Gaza and Wade Gaza. These soils can be found in the depression between the kurkar ridges of Dear El Balham Another transitional from is the sandy clay loam that have been covered by a layer (0.20-0.50m) of dune sand. This soil can be found east of Rafah and Khan Younis. Alluvial and Grumosolic are dominated by loamy clay texture are found on the slopes of the northern depressions between Beithanoun and Wadi Gaza. Borings east of el mortar ridge have revealed that alluvial deposits of a bout 25m in thickness occur

\[ S_{DL} = \frac{2}{3} F_r S_1 \]

\[ S_{DS} = \frac{2}{3} F_a S_s \]

As Gaza region classified as low seismcity it means that \( S_s < 0.25 \) from table (5.5), and Turkey is classified as high seismcity it means that \( 0.75g < S_s < 1.25g \). As \( S_{DS} \) is the important factor defining the \( C_S \) value (seismic response coefficient) and \( S_{DS} \) depending on \( F_a \) values.

5.3.3 Structural system factor

5.3.3.1 The structural systems of used on Turkey [23].

\( \text{a- Himis} \) is widely used structural type of building in the Eastern part of Turkey that is built by their resident without engineering considerations. Atypical Himis building is composed of thick perimeter walls and heavy roofs to provide heat isolation of the structure as shown in (Fig.5.8). The brittle behavior of the structural material that they are made of pours strength of the connection between members.
b- Unreinforced masonry buildings

Although their seismic performance is not good enough during earthquakes, unreinforced masonry buildings are preferred to reinforce masonry buildings in Turkey.

The common damage type was shear cracks due to the brittle behavior of the construction material. And in some of buildings infill walls were partially collapsed due to the lack of restraints in the out of plane direction this issue caused high level of nonstructural damage.

c- Reinforced concrete structures are the majority of the total structural stock in Turkey.

The reinforced concrete buildings are mostly composed of columns and beams and a few of them have shear walls. The commons type of failure was poor detailing at the critical region of beams columns and beam columns joints. On the other hand these connections had insufficient lap splice and transverse reinforcement. A few number of buildings had shear walls but in some cases due to insufficient transverse reinforcement wide shear cracks occurred in shear walls.

5.3.3.2 The structural system in Gaza Strip

a- Masonry bearing walls

This type of structures was used on regions which continue buildings as the refugee camps. The external walls and some interior walls are considered as bearing walls of unrinforced masonry buildings with reinforced slab and sometimes asbestos slabs. The usual floors construction consists of concrete supported by unreinforced masonry bearing walls. These buildings are very weak in resisting seismic loading.

b- Column and diaphragm system (skeleton structures)

This structural system is the main system which used widely on all Gaza strip. Up to six stories buildings are not constructed and designed for seismic loading. So that this system is considered week in resisting seismic loading, but the main parts of structural system which are accounted for resisting the lateral forced are:

a) The columns and diaphragm based on the fact that they can works as frames when loaded laterally.

b) The infill walls which are used as partitions these in Gaza are not designed to carry vertical loads.

c- Concrete frames

This type of structure is used on public buildings. As these buildings need big distance between columns, the frame system is considered the best solution for these buildings. The diaphragms are supported by reinforced concrete girders and columns. Moment resisting frame structure which are flexible, tend for have long natural periods.

From the above discussion it is clear that there is no big difference between the structural used on Turkey and Palestine except that for Himis structures. But the small difference on the factor is found due to many residential buildings were built without any engineering supervision and they are no controlled during there construction processes.
5.3.4 Construction quality

Among the observed damage buildings of Turkey after earthquakes, the common type of failure was the poor quality of concrete. The people do not prefer to use the ready mixed concrete just because it is too expensive. Instead they produce their own concrete by using the material they get from Murat river as aggregate.[29]

Another failure was poor detailing at the critical region of the structure elements, like insufficient amount of transverse reinforcement at the end region of beams, columns and beam column joint. On the other hand these connections has insufficient lap splice and transverse reinforcement. A few number of buildings had shear walls but in some cases due to insufficient transverse reinforcement and poor concrete quality wide shear cracks occurred in the shear walls.

The major reasons of the damage after earthquakes are the brittle behavior of the structural material that they are made of and poor strength of the connection between members. Also the high mass of the structure caused high lateral forces during seismic attack. On the other hand the weak connection of the braces between the members was not good enough to resist the lateral forces caused the total collapse of structure.

In some buildings infill walls were partially collapsed due to the lack of restraints in the out of plane direction. This issue caused high level of non structural damage like in Himis structures. Also many of the residential buildings were built without any engineering services and they are not controlled during their construction process. In the other side all the people in Palestine since 1990 are using ready mix concrete which is better than the hand mix in terms of quality and supervision. Also the aggregates used are a crushed stone which is very clean.

But the connection between the infill walls and the internal frame is poor and the bond between the hollow block pieces is also poor. In columns beams buildings there are adequate confinement (transverse reinforcement) in the vicinity of beam – column joint itself. Because of that Palestinian construction quality could be consider better than Turkish quality.

5.4 A new approach for evaluation Gaza buildings

Combining the modified Israel method and the statistical Turkish method help in solving all the deficiencies on these methods when applying to Gaza Strip buildings. This approach solves classification buildings degree to seismic hazard by creating and intermediate degree of hazard. The Turkish method based on observed damage and signification building attributes would provide more reliable and accurate results for regional assessments. The new approach focus on the structural system and configuration deficiencies including irregular geometry, concentration of mass or discontinuity in the lateral forced resisting system; minimum normalized lateral stiffness and minimum normalized lateral strength. In shortage on this new approach it is easy to evaluate the buildings accurately from all sides. The following subsections are a detailed description of the proposed method.

\[ S = S_o + \Delta S \]

*\( S_o \) is the basic score from Table (5.3) \( \Delta S \) is modifiers from Table (5.4)*

*\( Z = \text{peak ground acceleration} / \text{acceleration of gravity} \)*
Where Z:

<table>
<thead>
<tr>
<th>Value of Z</th>
<th>Risk Category</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25-0.30</td>
<td>H = High risk</td>
</tr>
<tr>
<td>0.15-0.20</td>
<td>M = Moderate risk</td>
</tr>
<tr>
<td>&lt; 0.1</td>
<td>L = Low moderate</td>
</tr>
</tbody>
</table>

Table 5.7 The basic structural score ($S_o$)

<table>
<thead>
<tr>
<th>Type of structure</th>
<th>Risk category</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood Frames</td>
<td>H</td>
</tr>
<tr>
<td>Steel moment-resisting frames</td>
<td>2.2</td>
</tr>
<tr>
<td>Braced steel frames</td>
<td>2</td>
</tr>
<tr>
<td>Concrete shear walls</td>
<td>2</td>
</tr>
<tr>
<td>Precast concrete large panels</td>
<td>1.5</td>
</tr>
<tr>
<td>Concrete frames</td>
<td>1</td>
</tr>
<tr>
<td>Precast concrete frames</td>
<td>0.5</td>
</tr>
<tr>
<td>Reinforced masonry</td>
<td>1.5</td>
</tr>
<tr>
<td>In filled frames</td>
<td>0.7</td>
</tr>
<tr>
<td>Plain brick / stone masonry</td>
<td>0.3</td>
</tr>
</tbody>
</table>

Table 5.8 Type of structure (Modifiers $\Delta S$)

<table>
<thead>
<tr>
<th>Types of structures</th>
<th>Modifiers $\Delta S$</th>
</tr>
</thead>
<tbody>
<tr>
<td>High-rise buildings (8 stories or more)</td>
<td>-0.5</td>
</tr>
<tr>
<td>Medium-rise buildings (4-7 stories)</td>
<td>0</td>
</tr>
<tr>
<td>Low – rise buildings (3 stories or less)</td>
<td>+0.3</td>
</tr>
<tr>
<td>Poor condition</td>
<td>-0.3</td>
</tr>
<tr>
<td>Poor condition of precast concrete structures</td>
<td>-0.5</td>
</tr>
<tr>
<td>Soft story</td>
<td></td>
</tr>
<tr>
<td>a- semi soft story</td>
<td>-0.2</td>
</tr>
<tr>
<td>b- complete soft story</td>
<td>-0.5</td>
</tr>
<tr>
<td>Significant eccentricity</td>
<td>-0.5</td>
</tr>
<tr>
<td>Pounding possible (for medium &amp; high rise building) Adjacent slab at same level.</td>
<td>-0.2</td>
</tr>
<tr>
<td>Pounding possible (for medium &amp; high rise building) Adjacent slab at different levels</td>
<td>-0.5</td>
</tr>
<tr>
<td>Heavy cladding (precast concrete or cut stone)</td>
<td>-0.5</td>
</tr>
<tr>
<td>short concrete columns</td>
<td>-0.5</td>
</tr>
<tr>
<td>year of construction : Before 1967</td>
<td></td>
</tr>
<tr>
<td>a- Poor conditions</td>
<td>-0.2</td>
</tr>
<tr>
<td>b- Good conditions</td>
<td>-0.05</td>
</tr>
<tr>
<td>1967-1980</td>
<td>0</td>
</tr>
<tr>
<td>After 1980</td>
<td></td>
</tr>
<tr>
<td>a- Poor conditions</td>
<td>+0.2</td>
</tr>
<tr>
<td>b- Good conditions</td>
<td>+0.5</td>
</tr>
<tr>
<td>Type of soil</td>
<td></td>
</tr>
<tr>
<td>S1 (Rock and stiff clay)</td>
<td>0</td>
</tr>
<tr>
<td>S2 (sand, gravel)</td>
<td>-0.2</td>
</tr>
<tr>
<td>S3 (soft and medium soil) or unknown</td>
<td>-0.3</td>
</tr>
<tr>
<td>S3 + (High-rise building)</td>
<td>-0.4</td>
</tr>
</tbody>
</table>
From this table we can find $\Delta S$

$S = S_0 + \Delta S$

If $S > 1$ $\Rightarrow$ Sufficient Seismic resistance
If $S < 1$ $\Rightarrow$ Insufficient Seismic resistance

We continue to find:

a- Minimum normalized lateral stiffens index

$$mnlstfi_{\text{min}} (I_{nx}, I_{ny})$$

(5.4)

$I_{nx}$ and $I_{ny}$ values in Eq. (5.1) are to be calculated by using Eq. (5.2).

$$I_{nx} = \frac{\sum (I_{col})_x + \sum (I_S)_x}{\sum A_j} \times 1000$$

$$I_{ny} = \frac{\sum (I_{col})_y + \sum (I_S)_y}{\sum A_j} \times 1000$$

(5.5)

Where;

$\Sigma (I_{col})_x$ and $\Sigma (I_{col})_y$ : summation of the moment of inertias of all columns about their centroidal $x$ and $y$ axes, respectively.

$\Sigma (I_S)_x$ and $\Sigma (I_S)_y$ : summation of the moment of inertias of all structural walls about their centroidal $x$ and $y$ axes, respectively.

$I_{nx}$ and $I_{ny}$ : total normalized moment of inertia of all members about $x$ and $y$ axes, respectively.

$\Sigma A_j$ : total story area above ground level.

b- Minimum normalized lateral strength index (MNLSI):

$$mnlsi = \min(A_{nx}, A_{ny})$$

(5.6)

Where,

$$A_{nx} = \frac{\sum (A_{col})_x + \sum (A_{sw})_x + 0.01 \sum (A_{mw})_x}{\sum A_j} \times 1000$$

$$A_{ny} = \frac{\sum (A_{col})_y + \sum (A_{sw})_y + 0.01 \sum (A_{mw})_y}{\sum A_j} \times 1000$$

(5.7)

For each column with a cross-sectional area denoted by $A_{col}$:

$$(A_{col})_x = k_x \times A_{col}$$

$$(A_{col})_y = k_y \times A_{col}$$

(5.8)

Where,

$k_x = 1/2$ for square and circular columns;
$k_x = 2/3$ for rectangular columns with $b_x > b_y$
$k_x = 1/3$ for rectangular columns with $b_x < b_y$; and
$k_y = 1 - k_x$. 
For each shear wall with cross-sectional area denoted by $A_{sw}$:

$$(A_{sw})_x = k_x \times A_{sw}$$

$$(A_{sw})_y = k_y \times A_{sw}$$  \hspace{1cm} (5.9)$$

**Where:**

$k_x = 1$ for structural walls in the direction of $x$-axis;

$k_x = 0$ for structural walls in the direction of $y$-axis; and

$k_y = 1 - k_x$.

For each unreinforced masonry filler wall with no window or door opening and having a cross sectional area denoted by $A_{mw}$:

c- Normalized redundancy score ($nrs$):

$$nrr = \frac{A_tr (n_{fx} - 1)(n_{fy} - 1)}{\sum A_{gf}} \times 1000$$  \hspace{1cm} (5.11)$$

**Where:**

$A_{tr}$: the tributary area for a typical column. $A_{tr}$ shall be taken as 25 m$^2$ if $n_{fx}$ and $n_{fy}$ are both greater than and equal to 3. In all other cases, $A_{tr}$ shall be taken as 12.5 m$^2$.

$n_{fx}$, $n_{fy}$: number of continuous frame lines in the critical story (usually the ground story) in $x$ and $y$ directions, respectively.

$A_{gf}$: the area of the ground story, i.e. the footprint area of the building. Depending on the value of $nrr$ computed from Eq. (8), the following discrete values are assigned to the normalized redundancy score ($nrs$):

$nrs = 1$ for $0 < nrr \geq 0.5$

$nrs = 2$ for $0.5 < nrr \geq 1.0$

$nrs = 3$ for $1.0 < nrr$

d- Soft story index ($ssi$):

$$SSI = \frac{H_1}{H_2}$$  \hspace{1cm} (5.12)$$

d- Overhang ratio ($or$):

$$or = \frac{A_{overhang}}{A_{gf}}$$  \hspace{1cm} (5.13)$$

$$DI_{L_s} = (0.620n - 0.246mnls i - 0.182mnlstf i - 0.699nrs + 3.269ssi + 2.728or - 4.905)$$  \hspace{1cm} (5.14)$$

In the case of immediate occupancy performance classification, the unstandardized discriminate function, where $DI_{IO}$ is the damage score corresponding to IOPC, based on these variables is:

$$DI_{IO} = (0.808n - 0.334mnls i - 0.107mnlstf i - 0.687nrs + 0.508ssi + 3.884or - 2.868)$$  \hspace{1cm} (5.15)$$

$$CF_{(lopc)} = (-0.090 n^3 + 1.498 n^2 - 7.518 n + 11.885)$$  \hspace{1cm} (5.16)$$

$$CF_{(lopc)} = (-0.085 n^3 + 1.416 n^2 - 6.951 n + 9.979)$$  \hspace{1cm} (5.16)$$
By comparing the \((CF)\) values with associated \((DI)\) value calculate performance grouping of the building for life safety performance classification (LSPC) and immediate occupancy performance classification (IOPC) as follows:

If \(DI_{LS} > CF_{LS}\) ⇒ Take \(PG_{LS} = 1\)
If \(DI_{LS} < CF_{LS}\) ⇒ Take \(PG_{LS} = 0\)
If \(DI_{Io} > CF_{LS}\) ⇒ Take \(PG_{Io} = 1\)
If \(DI_{Io} < CF_{LS}\) ⇒ Take \(PG_{Io} = 0\)

Table 5.9. Relationships among different classification criteria .

<table>
<thead>
<tr>
<th>Classification</th>
<th>Indicator Variable in LSPC</th>
<th>Indicator Variable in IOPC</th>
<th>Indicator Variable in Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>SAFE (None or light damage)</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>UNSAFE (Severe damage or collapse)</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>INTERMEDIATE</td>
<td>1</td>
<td>0</td>
<td>2</td>
</tr>
<tr>
<td>INTERMEDIATE</td>
<td>0</td>
<td>1</td>
<td>2</td>
</tr>
</tbody>
</table>

5.5 Summary
This new method has overcome the weaknesses found in the two evaluation methods that were adapted as a basics for the new approach. The proposed approach can be applied easily to the premises of the Gaza Strip buildings giving good results. For verifications, this method will be applied to many of the buildings of Gaza Strip to be classified in terms of resistance to earthquakes.
CHAPTER "6"

Application Of The New Approach To Various Buildings In Gaza Strip

Thirty three different samples of Gaza buildings were selected to be evaluated by the new method. These buildings are divided into five different categories.

**Category 1:** consists of twelve different skeleton type houses with three to five stories.

**Category 2:** consists of four tower buildings with eight to ten stories.

**Category 3:** consists of three residential villas with two stories.

**Category 4:** consists of six general frame buildings with four different schools and two different clinics.

**Category 5:** consists of eight different asbestos houses from different region Gaza Strip.

Sample building from each of five categories is taken with detailed calculation, in this chapter. The skeleton type house buildings has five with soft story and the others without. The residential villas have big distance between columns and walls to make big halls. Three towers buildings have shear walls in deferent locations and the fourth has not. The effect of earthquake magnitude, type of soil and the distance from center of earthquake is important in finding the cut of modification coefficient (CMC). At the end of this chapter a table contained detailed information about the sample with their classification for resisting earthquakes are developed.

6.1 Category 1: Infilled frames buildings (house no. 8)
This building was built on Rafah, it consists of three stories. There is not any soft story in the building. The soil type is soft clay.

1. Type of structure ($S_0$)
   The structural used is infilled frames; the grade should be (0.7)

2. Zone Factor According to seismic maps $Z = 0.25-0.30$ (high risk)

3. Modification factors:
   a - Poor condition the building is in a good case so the grade will be (0).
   b - Soft story, the building has no soft story the grade is (0).
   c - Significant eccentricity, the building is symmetrical so the grade should be (0).
   d - Ponding possible, So the grade is (-0.2).
   e - Year of construction, after 1980 the grade should be (+0.5).
   f - Type of soil, $S_3$ (sand and gravel) the grade should be (-0.3).

$$
\Delta s = 0.0 \\
S = S_0 + \Delta s \\
S = 0.7 \\
S < 1.0
$$

The building is classified insufficient E.Q resistance, so the calculation continues to find the stiffness and strength of columns and walls.

Total no of columns are 16

<table>
<thead>
<tr>
<th>Number of stories</th>
<th>$\sum$ floor area</th>
<th>$A_{col}$</th>
<th>$A_{sw}$</th>
<th>$\sum A_{mw}$</th>
<th>$I_{col (x)}$</th>
<th>$I_{col (y)}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>143</td>
<td>2.0</td>
<td>0</td>
<td>15.83</td>
<td>0.031</td>
<td>0.024</td>
</tr>
</tbody>
</table>

Minimum normalized lateral stiffness index ($mnlstfi$)

$$
I_{ns} = \frac{\sum (I_{col})_x + \sum (I_{sw})_x + \sum A_f}{1000}
$$

$mnlstfi = \text{Min} (I_x, I_y)$

$mnlstfi = 0.168$

Minimum normalized lateral strength index ($mnlsi$)

$$
A_{ns} = \frac{\sum (A_{col})_x + \sum (A_{sw})_x + 0.01 \sum (A_{mw})_x}{1000}
$$

$mnlsi = \text{min} (A_{nx}, A_{ny})$

$mnlsi = 5.26$

Normalized redundancy score ($nrs$)

$$
nrr = \frac{A_y (n_{fy} - 1)(n_{fy} - 1)}{\sum A_{sw}} \times 1000
$$

$nrs = 1.0$

Soft story index ($SSI$)

$$
ssi = \frac{H_1}{H_2} = 1.2
$$
f) overhang ratio (or):

\[
\text{or} = \frac{A_{\text{overhang}}}{A_{gf}} = 0.092
\]

\[
DI_{(LS)} = 0.62x + 0.246 \times mnlstfi - 0.082 \times mnlsl - 0.699 \times nrs + 3.269 \times ssi + 2.728 \times or - 4.905 = 0.0511
\]

\[
DI_{(IO)} = 0.808 \times n - 0.334 \times mnlstfi - 0.107 \times mnlsl - 0.687 \times nrs + 0.508 \times ssi + 3.884 \times or - 2.868 = 0.0249
\]

\[
CF_{(LSPC)} = 0.43
\]

\[
CF_{(IOPC)} = -0.609
\]

At magnitude 7.4, soft clay soil from table (2.11)

\[
CMC = 1.538
\]

\[
CF_{(LSPC)} \text{ modified} = CF_{LSPC} + ABS (CF_{LSPC}) \times (CMC-1) = 0.66
\]

\[
CF_{(IOPC)} = CF_{IOPC} + ABS (CF_{IOPC}) \times (CMC-1) = -0.28
\]

The building is classified as intermediate degree of hazard

Magnitude 7.0, soft clay soil CMC = 1.852 from table (2.10)

\[
CF_{LSPC} \text{ modified} = CF_{LSPC} + ABC (CF_{LSPC}), (CMC-1) = 0.79
\]

\[
CF_{IOPC} \text{ modified} = CF_{IOPC} + ABC (CF_{LSPC}), (CMC-1) = -0.09
\]

The building is classified as intermediate degree of hazard

Magnitude 6.50, soft clay soil CMC = 2.469 from table (2.9)

\[
CF_{LSPC} \text{ modified} = CF_{LSPC} + ABC (CF_{LSPC}), (CMC-1) = 1.06
\]

\[
CF_{IOPC} \text{ modified} = CF_{IOPC} + ABC (CF_{LSPC}), (CMC-1) = 0.285
\]

\[
CF_{LSPC} \text{ modified} > DI_{LS}
\]

\[
CF_{IOPC} \text{ modified} > DI_{IO}
\]

**Classification**: The building is classified as safe.

### 6.2 Category 2: High rise buildings (Tower no.1)
This tower building was built on Gaza city, it consists of ten stories. The ground has big halls which is classified as semi soft story. The structural system is infilled frame buildings with shear walls on the stairs and the left. The soil type is stiff sand.

1- Type of structure ($S_0$).

The structural used is unfilled frames; the grade should be (0.70).

2- Zone factor according to seismic maps $Z= 0.25-0.3$ (high risk).

3- Modification factors:
   a) The building is high rise buildings (10 stories), the grade should be (-0.5).
   b) Poor condition the building is in a good case so the grade will be (0).
   c) Significant eccentricity, the building is symmetrical so the grade should be (0).
   d) Soft story there is semi soft story so the grade should be (-0.2).
   e) Pounding possible the building is high rise with adjacent slab at different levels, so the grade is (0).
   f) Year of construction, after 1980 the grade should be (+0.5).
   g) Type of soil, $S_2$ (high rise building) the grade should be (-0.2).

$$\Delta s = - 0.4$$
$$S = S_0 + \Delta s$$
$$= 0.3 < 1.0$$

The building is classified insufficient E.Q resistance, so the calculation continues to find the stiffness and strength of columns and walls.

Total no of columns are 28.

<table>
<thead>
<tr>
<th>Number of stories</th>
<th>$\sum$ floor area</th>
<th>$A_{col} (x)$</th>
<th>$A_{sw}$</th>
<th>$A_{mw}$</th>
<th>$I_{col} (x)$</th>
<th>$I_{col} (y)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>560</td>
<td>6.08</td>
<td>5.6</td>
<td>16.43</td>
<td>0.164</td>
<td>0.157</td>
</tr>
</tbody>
</table>

To find minimum normalized lateral stiffness index

$$I_{nx} = \frac{\sum (I_{col})x + \sum (I_{sw})x}{\sum A_f} \times 1000$$

$$mnlstfi = \text{Min} (I_x, I_y)$$

$$mnlstfi = 1.73$$

Minimum normalized lateral strength index ($mnlsi$)

$$A_{nx} = \frac{\sum (A_{col})x + \sum (A_{sw})x + 0.01 \sum (A_{mw})x}{\sum A_f} \times 1000$$

$$A_{ny} = \frac{\sum (A_{col})y + \sum (A_{sw})y + 0.01 \sum (A_{mw})y}{\sum A_f} \times 1000$$

$$mnlsi = \min (A_{nx}, A_{ny}) = 80.2.$$ 

Normalized redundancy score ($nrs$)

Redundancy is the indication of the degree of the continuity of multiple frame lines which distribute lateral forces throughout the structural system.

$$nrr = \frac{A_f (n_{px} - 1)(n_{py} - 1)}{\sum A_{sf}} \times 1000$$

$$nrs = 1 \text{ for } 0 < nrr \leq 0.50.$$
Soft Story Index (SSI)

$$SSI = \frac{H_1}{H_2}$$

$H_1$ : ground Story height  
$H_2$ : the height of first story

SSI = 1.33

f) Overhang ratio (or):-
Summation of the overhang area of each story divided by the area of ground story $A_gf$.

$$or = \frac{A_{\text{overhang}}}{A_g} = 0.19$$

$$DI_{(ls)} = 0.62 \times n - 0.246 \times mnlstfi - 0.082 \times mnlsi - 0.699 \times nrs + 3.269 \times ssi + 2.728 \times or - 4.905 = 3.576$$

$$Di_{(Io)} = 0.808 \times n - 0.334 \times mnlstfi - 0.107 \times mnlsi - 0.687 \times nrs + 0.508 \times ssi + 3.884 \times or - 2.868 = 4.5$$

Cut off function based on number of stories

$$CF_{(lspc)} = -0.09 n^3 + 1.498 n^2 - 7.518n + 11.885 = -3.495$$

$$CF_{(IOPC)} = 0.085 n^3 - 1.416n^2 + 6.951n + 9.979 = -2.931$$

To find modification coefficients (CMC)

Stiff soil $V_s$ (201-400) m/s, $M_w=7.4$ from table (2.11)

$$CMC = 2.414$$

$$CF_{LSPC \text{ modified}} = CF_{LSPC} + ABC (CF_{LSPC}) (CMC-1) = +1.44$$

$$CF_{IOPC \text{ modified}} = CF_{IOPC} + ABS (CF_{IOPC}) (CMC-1) = 1.213$$

$$CF_{LSPC \text{ modified}} < DI_{LS}$$

$$CF_{IOPC \text{ modified}} < DI_{IO}$$

The building is classified as unsafe at $M_w = 7.4$ from table (2.11)

At $M_w = 7.0$, stiff soil $V_s$ = (201-400) m/s from table (2.10).

$$CMC = 2.939$$

$$CF_{LSPC \text{ modified}} = CF_{LSPC} + ABC (CF_{LSPC}) (CMC-1) = 3.28$$

$$CF_{IOPC \text{ modified}} = CF_{IOPC} + ABC (CF_{LSPC}) (CMC-1) = 2.75$$

$$CF_{LSPC \text{ modified}} < DI_{LS}.$$  

$$CF_{IOPC \text{ modified}} < DI_{IO}.$$  

The building is classified as unsafe at $M_w = 7.0$.

At $M_w = 6.50$, stiff soil $V_s$ = (201-400) m/s from table (2.9)

$$CMC = 4.033$$

$$CF_{LSPC \text{ modified}} = CF_{LSPC} + ABC (CF_{LSPC}) (CMC-1) = 7.10$$

$$CF_{IOPC \text{ modified}} = CF_{IOPC} + ABC (CF_{LSPC}) (CMC-1) = 5.95$$

$$CF_{LSPC \text{ modified}} > DI_{LS}$$

$$CF_{IOPC \text{ modified}} > DI_{IO}$$

Classification: The building is classified as safe at $M_w = 6.50$. 

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6.3 Category 3 : Residential villas (Villa no. 1):

This villa was built on Rafah, it consists of tow stories with big halls. The soil type is stiff sand and structural system is infilled frame system.

1- Type of structure (S₀).
   The structural used is infilled frames; the grade should be (0.7).

2- Zone factor according to seismic maps Z = 0.25-0.30 (high risk).

3- Modification factors:
   a) The building is low rise building, the grade will be (+0.3).
   b) Poor condition the building is in a good case so the grade will be (0).
   c) Soft story, the building has no soft story the grade is (0).
   d) Significant eccentricity, the building is unsymmetrical so the grade should be (-0.5).
   e) Pounding possible, so the grade is (-0.2).
   f) Year of construction, after 1980 the grade should be (+0.5).
   g) Type of Soil, S₂ (sand and gravel) the grade should be (-0.2).

\[ \Delta s = +0.1 \]
\[ S = S_0 + \Delta s. \]
\[ S = 0.8. \]
\[ S < 1 \]

The building is classified insufficient E.Q resistance, so the calculation continues to find the stiffness and strength of columns and walls.
Total no of columns are 17.

<table>
<thead>
<tr>
<th>Number of stories</th>
<th>(\sum) floor area</th>
<th>(A_{col})</th>
<th>(A_{sw})</th>
<th>(A_{mw})</th>
<th>(I_{col(x)})</th>
<th>(I_{col(y)})</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>215</td>
<td>2.08</td>
<td>0</td>
<td>19</td>
<td>0.017</td>
<td>0.034</td>
</tr>
</tbody>
</table>

Minimum normalized lateral stiffness index (\(\text{mnlstfi}\))

\[
I_{nx} = \frac{\sum (I_{col})_x + \sum (I_{sw})_x + \sum A_f}{\sum A_f} \times 1000
\]

\(\text{mnlstfi} = \text{Min} (I_x, I_y) = 0.08\)

Minimum normalized lateral strength index (\(\text{mnlsi}\))

\[
A_{nx} = \frac{\sum (A_{col})_x + \sum (A_{sw})_x + 0.01 \sum (A_{mw})_x}{\sum A_f} \times 1000
\]

\(\text{mnlsi} = \min (A_{nx}, A_{ny}) = 3.70\)

Normalized redundancy score (\(\text{nrs}\))

\[
nrr = \frac{A_y (n_{fy} - 1)(n_{fy} - 1)}{\sum A_{fy}} \times 1000
\]

\(\text{nrs} = 1.0\)

Soft story index (\(\text{ssi}\))

\[
SSI = \frac{H_1}{H_2} = 1.20
\]

f) Overhang ratio (\(\text{or}\)):-

\[
\text{or} = \frac{A_{overhang}}{A_{gf}} = 0.025
\]

\[
DI_{(LS)} = 0.62 \times n - 0.246 \times \text{mnlstfi} - 0.082 \times \text{mnlsi} - 0.699 \times nrr + 3.269 \times ssi + 2.728 \times or - 4.905 = 1.06
\]

\[
DI_{(IO)} = 0.808 \times n - 0.334 \times \text{mnlstfi} - 0.107 \times \text{mnlsi} - 0.687 \times nrr + 0.508 \times ssi + 3.884 \times or - 2.868 = 1.65
\]

\(\text{CF}_{LS} = 2.84\)

\(\text{CF}_{IO} = -1.76\)

Stiff soil \(V_s, (201-400) \text{ m/s} \ M_w=7.4\) from table (2.11)

\[
\text{CMC} = 2.414
\]

\[
\text{CF}_{(Lspc)\text{modified}} = \text{CF}_{Lspc} + \text{ABS} (\text{CF}_{Lspc}) (\text{CMC}-1) = 6.85
\]

\[
\text{CF}_{(IOPC)} = \text{CF}_{IOPC} + \text{ABS} (\text{CF}_{IOPC}) (\text{CMC}-1) = 0.728
\]

\[
\text{CF}_{LSPC\text{modified}} \geq DI_{LS}
\]

\[
\text{CF}_{IOPC\text{modified}} \geq DI_{IO}
\]

**Classification:** The building is classified as sufficient degree of earthquake resistance.
6.4 Category 4: School buildings (School no.1)

This school building was built in Beit Lahya, it consists of three stories each story with twelve classes and to stairs. The soil type is soft clay, and the structural system was frame building.

1- Type of structure (S₀)
   The structural used is concrete frames; the grade should be (1).

2- Zone factor according to seismic maps Z= 0.25-0.3 (high risk).

3- Modification factors:
   a) The building is low rise buildings (3 stories), the grade should be (+0.3).
   b) Poor condition the building is in good case so the grade will be (0).
   c) Soft story there is not any soft story floor the grade should be (0).
   d) Year of construction, after 1980 the grade should be (+0.5).
   e) Type of Soil, S₃ (soft and medium soil) the grade should be (-0.3).

\[ \Delta s = +0.5 \]
\[ S = S_0 + \Delta s. \]
\[ = 1.5 >1. \]

The building is classified sufficient Eq resistance, its supposed to stop as the approach but the calculation is continue to confirm the results.

Total no of columns are 18.

<table>
<thead>
<tr>
<th>Number of stories</th>
<th>( \sum ) floor area</th>
<th>( A_{col(x)} )</th>
<th>( A_{sw} )</th>
<th>( A_{mw} )</th>
<th>( I_{col(x)} )</th>
<th>( I_{col(y)} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>292</td>
<td>2.025</td>
<td>0</td>
<td>25.44</td>
<td>0.035</td>
<td>0.011</td>
</tr>
</tbody>
</table>

To find minimum normalized lateral stiffness index (mnlstfι)
\[ I_{nx} = \sum (I_{col}) x + \sum (I_{rv}) x + \sum A_y \times 1000 \]

\[ mnlstfi = \min (I_x, I_y) = 0.038 \]

**Minimum normalized lateral strength index (mnlsi)**

\[ A_{nx} = \sum (A_{col}) x + \sum (A_{rv}) x + 0.01 \sum (A_{mv}) x \times 1000 \]

\[ mnlsi = \min (A_{nx}, A_{ny}) = 2.80 \]

**Normalized redundancy score (nrs)**

\[ nrr = \frac{A_u (n_{xv} - 1)(n_{yv} - 1)}{\sum A_{sf}} \times 1000 \]

\[ nrs = 1.0 \]

**Soft story index (SSI)**

\[ SSI = \frac{H_1}{H_2} = 1.20 \]

f) **Overhang ratio (or):**

\[ or = \frac{A_{overhang}}{A_{gf}} = \frac{61.65}{299} = 0.205 \]

\[ DI_{(LS)} = 0.62 \times n - 0.246 \times mnlstfi - 0.082 \times mnlsi - 0.699 \times nrs + 3.269 \times ssi + 2.728 \times or - 4.905 = 0.23 \]

\[ DI_{(IO)} = 0.808 \times n - 0.334 \times mnlstfi - 0.107 \times mnlsi - 0.687 \times nrs + 0.508 \times ssi + 3.884 \times or - 2.868 = -0.18 \]

**Cut off function based on number of stories.**

\[ CF_{(Lspc)} = -0.09 \times n^3 + 1.498 \times n^2 - 7.518 \times n + 11.885 = 0.383. \]

\[ CF_{(Iopc)} = 0.085 \times n^3 + 1.416 \times n^2 - 6.951 \times n + 9.979 = -0.425 \]

To find modification coefficients (CMC) for \( M_w = 7.4 \) from table (2.11)

stiff soil \( \text{CMC} = 2.414 \)

\[ CF_{(Lspc) \text{ modified}} = CF_{Lspc} + \text{ABS} (CF_{Lspc}) (CMC-1) = 0.924 \]

\[ CF_{(Iopc) \text{ modified}} = CF_{Iopc} + \text{ABS} (CF_{Iopc}) (CMC-1) = 0.17. \]

\[ CF_{LSPC \text{ modified}} > DI_{LS} \]

\[ CF_{IOPC \text{ modified}} > DI_{IO} \]

**Classification:** The building is classified as safe at \( M_w = 7.4 \).
6.5 Category 5: Asbestos shelter buildings (Asbestos house no.6)

This building was built on Khanyonis refugees camp, it consist of on story. The walls are bearing walls and the slab is sheet cover from asbestos. The soil type is stiff sand.

1- Type of structure \((S_o)\).
   The structural used in plane brick / stone masonry, so the grade should be \((0.30)\).
2- Zone factor in Gaza according to seismic maps \(Z = 0.25 - 0.3\) (high risk).
3- Modification factors:
   a- The building is low rise buildings, the grade should be \((+0.3)\).
   b- Poor condition the building is medium case there is some hair cracks the grade will be \((-0.3)\).
   c- Significant eccentricity, the building is un symmetrical so the grade should be \((-0.5)\).
   d- Year of construction, 1967-1980 the grade should be \((0)\).
   e- Type of Soil, \(S_2\) (soft and medium soil) or unknown the grade should be \((-0.2)\)

\[
\Delta s = -0.7.
S = S_0 + \Delta s.
= -0.4.
S < 1.0.
\]

The building is classified insufficient E.Q resistance, so the calculation continues to find the stiffness and strength of columns and walls.

Total no of columns are \((0)\).

<table>
<thead>
<tr>
<th>Number of stories</th>
<th>(\sum \text{floor area})</th>
<th>(A_{\text{col(x)}})</th>
<th>(A_{\text{sw}})</th>
<th>(A_{\text{mw}})</th>
<th>(I_{\text{col(x)}})</th>
<th>(I_{\text{col(y)}})</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>99</td>
<td>0</td>
<td>0</td>
<td>11.13</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Minimum normalized lateral stiffness index \((\text{mnlstfi})\).
\[ I_{ny} = 0 \]
\[ mnlsfi = 0 \]

**Minimum normalized lateral strength index (mnlsi)**

\[ A_{nx} = \sum (A_{col})x + \sum (A_{sw})x + 0.01\sum (A_{mw})x \times 1000 \sum A_f \]

\[ mnlsi = \min (A_{nx}, A_{ny}) = 0.56 \]

**Normalized redundancy score (nrs)**

\[ nrr = \frac{A_n(n_f - 1)}{\sum A_{gf}} \times 1000 \]

\[ nrs = 1.0 \]

**Soft story index (SSI)**

\[ SSI = \frac{H_1}{H_2} = 1.20 \]

f) **Overhang ratio (or):**

\[ or = \frac{A_{overhang}}{A_{gf}} = 0 \]

\[ DI_{(ux)} = 0.62n - 0.246mnlsfi - 0.082mnlsi - 0.699nrs + 3.269SSI + 2.728\times or - 4.905 = -1.06 \]

\[ DI_{(vo)} = 0.808n - 0.334mnlsfi - 0.107mnlsi - 0.687nrs + 0.508SSI + 3.884\times or - 2.868 = -1.611 \]

At \( M = 7.4 \), \( V_s = (0-200) \), \( CMC = 1.536 \)

\[ CF_{(Lspc)} = -5.77 \quad \quad CF_{(Iopc)} = -4.359 \]

To find modification 7.4, soft clay soil from table (4.7)

**CMC = 1.538**

\[ CF_{(Lspc)\text{modified}} = CF_{Lspc} + ABS(CF_{Lspc})(CMC-1) = -2.66 \]

\[ CF_{(Iopc)\text{modified}} = CF_{Iopc} + ABS(CF_{Iopc})(CMC-1) = -2.013 \]

\[ CF_{LSPC\text{modified}} < DI_{LS} \]
\[ CF_{IOPC\text{modified}} < DI_{IO} \]

**Classification**: The building is classified as insufficient degree of resistance.
### 6.6 Results of applying the new approach to evaluate sample buildings (Gaza strip)

<table>
<thead>
<tr>
<th>N o.</th>
<th>Building type</th>
<th>Location</th>
<th>Number of Stories</th>
<th>Soil type</th>
<th>Soft story</th>
<th>Magnitude (7.4)</th>
<th>Magnitude (7.6)</th>
<th>Magnitude (6.5)</th>
<th>Magnitude (6.0)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Safe Intermediate Unsafe</td>
<td>Safe Intermediate Unsafe</td>
<td>Safe Intermediate Unsafe</td>
<td>Safe Intermediate Unsafe</td>
</tr>
<tr>
<td>1</td>
<td>House (no. 1)</td>
<td>Gaza</td>
<td>3</td>
<td>Stiff Soil</td>
<td>yes</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>House (no. 2)</td>
<td>Gaza</td>
<td>3</td>
<td>Stiff Soil</td>
<td>No</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>House (no. 3)</td>
<td>Rafah</td>
<td>3</td>
<td>Soft Clay</td>
<td>No</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>House (no. 4)</td>
<td>Rafah</td>
<td>4</td>
<td>Stiff Soil</td>
<td>No</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>House (no. 5)</td>
<td>Khan younis</td>
<td>4</td>
<td>Soft Clay</td>
<td>Semi Soft story</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>House (no. 6)</td>
<td>Khan younis</td>
<td>4</td>
<td>Stiff Soil</td>
<td>No</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>House (no. 7)</td>
<td>Gaza</td>
<td>4</td>
<td>Stiff Soil</td>
<td>No</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>House (no. 8)</td>
<td>Rafah</td>
<td>4</td>
<td>Soft Clay</td>
<td>No</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>House (no. 9)</td>
<td>Khan younis</td>
<td>4</td>
<td>Soft Clay</td>
<td>No</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>House (no. 10)</td>
<td>Gaza</td>
<td>4</td>
<td>Soft Clay</td>
<td>No</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>House (no. 11)</td>
<td>Rafah</td>
<td>4</td>
<td>Soft Clay</td>
<td>No</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>House (no. 12)</td>
<td>Rafah</td>
<td>4</td>
<td>Soft Clay</td>
<td>No</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Category 2: Tower building

<table>
<thead>
<tr>
<th>N o.</th>
<th>Building type</th>
<th>Location</th>
<th>Number of Stories</th>
<th>Soil type</th>
<th>Magnitude (7.4)</th>
<th>Magnitude (7.6)</th>
<th>Magnitude (6.5)</th>
<th>Magnitude (6.0)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Safe Intermediate Unsafe</td>
<td>Safe Intermediate Unsafe</td>
<td>Safe Intermediate Unsafe</td>
<td>Safe Intermediate Unsafe</td>
</tr>
<tr>
<td>1</td>
<td>Tower (no. 1)</td>
<td>Gaza</td>
<td>10</td>
<td>Semi Soft story</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Towers (no. 2)</td>
<td>Gaza</td>
<td>8</td>
<td>Semi Soft story</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Tower (no. 3)</td>
<td>Gaza</td>
<td>8</td>
<td>Semi Soft story</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Tower (no. 4)</td>
<td>Rafah</td>
<td>8</td>
<td>Semi Soft story</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Category 3: Residential Villas

<table>
<thead>
<tr>
<th>N o.</th>
<th>Building type</th>
<th>Location</th>
<th>Number of Stories</th>
<th>Soil type</th>
<th>Magnitude (7.4)</th>
<th>Magnitude (7.6)</th>
<th>Magnitude (6.5)</th>
<th>Magnitude (6.0)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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<td></td>
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<td>Safe Intermediate Unsafe</td>
<td>Safe Intermediate Unsafe</td>
<td>Safe Intermediate Unsafe</td>
</tr>
<tr>
<td>1</td>
<td>Villa (no. 1)</td>
<td>Rafah</td>
<td>2</td>
<td>Stiff Soil</td>
<td>No</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Villa (no. 2)</td>
<td>Nusseral</td>
<td>2</td>
<td>Stiff Soil</td>
<td>No</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Villa (no. 3)</td>
<td>Gaza</td>
<td>2</td>
<td>Stiff Soil</td>
<td>No</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### 6.6 Results of applying the new approach to evaluate sample buildings (Gaza strip)

<table>
<thead>
<tr>
<th>N. S.</th>
<th>Building type</th>
<th>Location</th>
<th>Number of Stories</th>
<th>Soil type</th>
<th>Soft story</th>
<th>Magnitude (7.4)</th>
<th>Magnitude (7.6)</th>
<th>Magnitude (6.5)</th>
<th>Magnitude (6.0)</th>
</tr>
</thead>
<tbody>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>safe</td>
<td>Intermediate</td>
<td>Unsafe</td>
<td>safe</td>
</tr>
<tr>
<td>1</td>
<td>School (no. 1)</td>
<td>Beit lahya</td>
<td>3</td>
<td>Soft Clay</td>
<td>No</td>
<td>✔</td>
<td></td>
<td>✔</td>
<td>✔</td>
</tr>
<tr>
<td>2</td>
<td>School (no. 2)</td>
<td>Gaza</td>
<td>3</td>
<td>Soft Clay</td>
<td>No</td>
<td>✔</td>
<td></td>
<td>✔</td>
<td>✔</td>
</tr>
<tr>
<td>3</td>
<td>School (no. 3)</td>
<td>Beit hanoon</td>
<td>3</td>
<td>Soft Clay</td>
<td>No</td>
<td>✔</td>
<td></td>
<td>✔</td>
<td>✔</td>
</tr>
<tr>
<td>4</td>
<td>School (no. 4)</td>
<td>Rafah</td>
<td>3</td>
<td>Soft Clay</td>
<td>No</td>
<td>✔</td>
<td></td>
<td>✔</td>
<td>✔</td>
</tr>
<tr>
<td>5</td>
<td>Clinic (no. 1)</td>
<td>Nusseirat</td>
<td>3</td>
<td>Soft Clay</td>
<td>No</td>
<td>✔</td>
<td></td>
<td>✔</td>
<td>✔</td>
</tr>
<tr>
<td>6</td>
<td>Clinic (no. 2)</td>
<td>Gaza</td>
<td>3</td>
<td>Soft Clay</td>
<td>No</td>
<td>✔</td>
<td></td>
<td>✔</td>
<td>✔</td>
</tr>
</tbody>
</table>

### Category 4: General Building

<table>
<thead>
<tr>
<th>N. S.</th>
<th>Building type</th>
<th>Location</th>
<th>Number of Stories</th>
<th>Soil type</th>
<th>Soft story</th>
<th>Magnitude (7.4)</th>
<th>Magnitude (7.6)</th>
<th>Magnitude (6.5)</th>
<th>Magnitude (6.0)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>safe</td>
<td>Intermediate</td>
<td>Unsafe</td>
<td>safe</td>
</tr>
<tr>
<td>1</td>
<td>House (no. 1)</td>
<td>Rafah</td>
<td>1</td>
<td>Stiff Soil</td>
<td>-</td>
<td>✔</td>
<td></td>
<td>✔</td>
<td>✔</td>
</tr>
<tr>
<td>2</td>
<td>House (no. 2)</td>
<td>Rafah</td>
<td>1</td>
<td>Soft Clay</td>
<td>-</td>
<td>✔</td>
<td></td>
<td>✔</td>
<td>✔</td>
</tr>
<tr>
<td>3</td>
<td>House (no. 3)</td>
<td>Rafah</td>
<td>1</td>
<td>Stiff Soil</td>
<td>-</td>
<td>✔</td>
<td></td>
<td>✔</td>
<td>✔</td>
</tr>
<tr>
<td>4</td>
<td>House (no. 4)</td>
<td>Khan younis</td>
<td>1</td>
<td>Soft Clay</td>
<td>-</td>
<td>✔</td>
<td></td>
<td>✔</td>
<td>✔</td>
</tr>
<tr>
<td>5</td>
<td>House (no. 5)</td>
<td>Khan younis</td>
<td>1</td>
<td>Stiff Soil</td>
<td>-</td>
<td>✔</td>
<td></td>
<td>✔</td>
<td>✔</td>
</tr>
<tr>
<td>6</td>
<td>House (no. 6)</td>
<td>Khan younis</td>
<td>1</td>
<td>Stiff Soil</td>
<td>-</td>
<td>✔</td>
<td></td>
<td>✔</td>
<td>✔</td>
</tr>
<tr>
<td>7</td>
<td>House (no. 7)</td>
<td>Gaza</td>
<td>1</td>
<td>Soft Clay</td>
<td>-</td>
<td>✔</td>
<td></td>
<td>✔</td>
<td>✔</td>
</tr>
<tr>
<td>8</td>
<td>House (no. 8)</td>
<td>Gaza</td>
<td>1</td>
<td>Soft Clay</td>
<td>-</td>
<td>✔</td>
<td></td>
<td>✔</td>
<td>✔</td>
</tr>
</tbody>
</table>
6.7 Descriptive drawings of the results of applying the new approach.

Fig. 6.1 Percentage results of evaluation (skelton type)

Fig. 6.2 Percentage results of evaluation (Tower type)
Fig. 6.3 Percentage results of evaluation (Villa type)

Fig. 6.4 Percentage results of evaluation (general building type)

Fig. 6.5 Percentage results of evaluation (Asbestos type)
6.8 Discussion of results

It is clear from results that the skeleton type system is classified as sufficient earthquake resistant at high magnitude (7.4). However, the problem appears on this system when the building has soft story, the result is insufficient earthquake resistance at (7.4, 7.0, and 6.5) and sufficient magnitude (6.0). This means that all the buildings with soft story need strengthening to resist earthquake.

Tower buildings which have more than eight stories are classified safe if it has enough shear walls, but in case of there are not sufficient shear walls, the results were insufficient earthquake resistance at magnitude (7.4, 7.0, and 6.5) and sufficient at magnitude (6.0). The big problem appeared at the tower building which hasn’t shear walls. The results were insufficient at magnitude (7.4, 7.0, and 6.5) and moderate at magnitude (6.0). To know that if the shear walls are enough on any buildings, it is possible through calculation the discriminate function from this approach, the damage score should be less than the cutoff values.

General frame buildings which are designed as concrete frame system are considered suitable system of structural systems used in Gaza Strip. Evaluation results by the new approach are sufficient earthquake resistance at high magnitude (7.4). It means that these buildings with frames have good earthquake resistance at high earthquakes magnitude.

Residential villas normally consist of two stories, so the effect of lateral earthquake loads is small. These buildings are considered safe on resisting seismic loading that is from applying the new approach of evaluation.

Asbestos houses are evaluated by the new approach as unsafe at all magnitudes (7.4, 7.6, 5, and 6) which is logic. In all evaluation methods, the asbestos houses are evaluated as insufficient earthquake resistance and the houses need strengthening.
CHAPTER 7
Conclusions And Recommendations

7.1 Achievement
The proposed evaluation approach has been applied on 33 buildings of various types. It is found that using the new evaluation approach has results in a better seismic evaluation. Evaluation process started by making rapid screening for the buildings, then a detailed evaluation was done by discriminate function, and finally the classification of the building is completed. Thus, the proposed approach has been verified using actual cases.

7.2 Conclusions
A detailed seismic evaluation of structural system of existing building needs to be performed to determine the nature and extent of deficiencies which can cause poor performance in future earthquakes. This evaluation also helps to decide whether structural modifications are required at few locations in the structure for deficient components only or interventions are needed at the structure level so that its global behavior is improved and thus seismic demands on components are reduced.

There is no contrast in evaluation of Gaza buildings between the Israeli method and the new approach method. But the new method gives us detailed evaluations to all the members which resist lateral forces for earthquakes, columns, beams, shear walls and infill walls. Also the method solved the Israeli method problems on evaluation results instead of evaluating the building as sufficient or insufficient earthquake resistance, while the second results mix between buildings which unsafe or only need some strengthening techniques to improve its ability to resist seismic lateral loads by increasing the strength stiffness and/or ductility. Its is recommended to make a rapid screening by the new approach and if the results are sufficient earthquake resistance, the evaluation process will be stopped and the building is classified as safe and has sufficient earthquake resistance. If the results give insufficient, the evaluation process continues through a detailed calculation to all the members which resist earthquake, till the evaluation process finished. If the results are sufficient the building has sufficient earthquake resistance and if the results are moderate, strengthening techniques are necessary to increase its ability to resist earthquake. Finally if the results are insufficient then the building is unsafe, and the recommendation should be demolishing for the building.

7.3 Recommendations
The followings are some important recommendations proposed to be used in Gaza buildings where found from the application of a new approach method.

7.3.1 Skeleton Type (Unfilled frame building)
This structure system is the main system which used widely on all Gaza strip. This system is considered as has sufficient earthquake resistance at high magnitude (7.4), but the problem appeared on the system, when the building has soft story, the results are insufficient earthquake resistance at (7.4, 7.0, and 6.5) and sufficient at magnitude (6.0). The column area has big affect on reducing the damage index, so therefore the classification buildings will have sufficient earthquake resistance. The recommendations to avoid weakness in this system are:

1- The design of skeleton type buildings should conform to requirements of earthquakes design. This should cover soft story, irregularities and soil strength.
2- The existing buildings may be strengthened in accordance with known
strengthening techniques. These may include jacketing of members, adding
new members, soil injection to strengthen the soil and improving the connections
between the resisting elements by steel elements, etc.

7.3.2 Tower buildings
These building are used on Gaza strip with percentage of about 1% from total
buildings on Gaza strip. These buildings are considered moderate to safe on resisting
seismic loading from the result of applying new approach of evaluation. Some of the
tower building are consider safe if the building has enough share walls, but if it hasn’t
sufficient shear walls the results was insufficient earthquake resistance at magnitude
(7.4, 7.0, and 6.5) and sufficient at magnitude (6.0). The problem appeared at the tower
which hasn’t share walls, the results were insufficient earthquake resistance at high
magnitude. To avoid the weakness on resisting earthquakes loads the same
recommendation which used before on in filled frame buildings are used and the new
recommendations are:-

1- The design of tower buildings type should conform to requirements of
earthquakes design. This should cover soft story, irregularities, soil strength,
enough shear walls.
2- The existing tower type buildings may be strengthen in accordance with known
techniques. These may include jacketing of members, adding new members, and soil
injection to strength the soil, and adding new shear walls that leads to an
increase in the global stiffness and strength of structures.

7.3.3 Residential villas
These buildings always consists of two stories, so the effect of lateral earthquake
loads is little and these buildings are considered safe on resisting seismic loading from
the results of applying the new method, but the only problem is the big distance
between columns which is used on these buildings to make big halls, the same
recommendation which are used before on unfilled frame, tower buildings are used on
these buildings and the new recommendation which is different is:

1- The design of Residential villas type buildings should conform to
requirements of earthquakes design. This should includes using drop beams
on big distance or increasing the diaphragm thickness to avoid shear failure.
2- The existing villas may be strengthen in accordance with known techniques
these may include jacketing of members, adding new members, and soil
injection

7.3.4 General frame buildings
These building are designed as concrete frame system which considered as
suitable for these buildings. The results are sufficient earthquake resistance at high
magnitude (7.4) the diaphragms are supported by concrete girders and columns. Lateral
forces are resisted by the frame and the infill masonry walls. These building are
consisting of two to four stories and are considered safe by all the evaluation method
and the recommendations which are used hear are:-
1. The design of general frame buildings should conform to requirements of earthquakes design. This should cover suitable relative strengths column /beam , beam /column joint ties and restraining of columns.

2. The existing buildings may be strengthen in accordance with known techniques . these may include jacketing of members , adding new members , and soil injection.

### 7.3.5 Asbestos houses

These types of buildings were used in regions which were built in the past in refuges camps. The external walls and some interior walls are bearing walls of unreinforced masonry with asbestos sheet cover. By the new approach of evaluation, these buildings are classified as insufficient earthquake resistance and unsafe. To confirm the results the calculation is continue through finding the stiffness and the strength of walls and columns, and the results was insufficient earthquake resistance. The recommendation for old asbestos house is demolishing and replacement with concrete buildings.
References:


7- Bungales, S. Taranath., "Wind and earthquake resistant buildings". Structural analysis and design, CRS Publisher ,15-12-2004  892pp.


26- حساب القوى الزلزالية الأفقية المكافئة التي تتعرض لها المباني . د . جلال الدبيك مدير مركز علوم الأرض و هندسة الزلزال ، جامعة النجاح.


# Appendix A

Samples for skeleton type (infill frame building)

## 1- House (no.1)

![Diagram of House (no.1)](image1)

## 2- House (no.2)

![Diagram of House (no.2)](image2)
3- House (no.3)

4- House (no.4)
Sample for tower buildings

<table>
<thead>
<tr>
<th>1- Tower (no. 1)</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image1" alt="Tower Diagram" /></td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>2- Tower (no. 2)</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image2" alt="Tower Diagram" /></td>
</tr>
</tbody>
</table>
3- Tower (no. 3)

4- Tower (no. 4)
Sample for **Residential Villas**

1- Villa (no. 1)

2- Villa (no. 2)
3- Villa (no. 3)
Sample for general buildings

1- School (no. 1)

2- School (no. 2)
5- Clinic (no. 1)

6- Clinic (no. 2)
Sample for asbestos houses

1- House (no. 1)

2- House (no. 2)
5- House (no. 5)

6- House (no. 6)
Appendix B

Tables for Canadianian evaluation method

Table B.1 Effect of seismicity (NRC, 1993)

<table>
<thead>
<tr>
<th>A</th>
<th>Seismicity</th>
<th>Design NBCC</th>
<th>Effective Seismic Zone (Z_v or Z_v+1 if Z_a &gt; Z_v)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>Per 65</td>
<td>1.0</td>
<td>1.5</td>
<td>2.0</td>
</tr>
<tr>
<td>65-85</td>
<td>1.0</td>
<td>1.0</td>
<td>1.3</td>
</tr>
<tr>
<td>Post85</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>

Table B.2. Effect of type of structures (NRC, 1993)

<table>
<thead>
<tr>
<th>C</th>
<th>Type of structure</th>
<th>Design NBCC</th>
<th>Construction Type and symbol</th>
<th>Concrete</th>
<th>Precast</th>
<th>MI</th>
<th>Masonary</th>
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<tbody>
<tr>
<td></td>
<td>Per-70</td>
<td>1.2</td>
<td>2.0</td>
<td>1.0</td>
<td>1.2</td>
<td>1.5</td>
<td>2.0</td>
</tr>
<tr>
<td></td>
<td>70-90</td>
<td>1.2</td>
<td>2.0</td>
<td>1.0</td>
<td>1.2</td>
<td>1.5</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>Post 90</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>

MI* = Masonry Infill
WLF = Wood Light
WPB = Wood, Post and Beam
SLF = Steel Light frame
SMF = Steel Moment frame
SBF = Steel Braced Frame
SCW = Steel frame with Concrete shear Wall
CMF = Concrete Moment Frame
CSW = Concrete Shear Walls
PCF = Precast Concrete Frame
PCW = Precast Concrete Walls
SIW = Steel frame with Infill masonry shear Walls
CIW = Concrete frame with Infill masonry shear Walls
RML = Reinforced Masonry bearing walls with wood or metal deck floors or roofs
RMC = Reinforced Masonry bearing walls with Concrete diaphragms
URM = Unreinforced Masonry bearing walls building

Table B.3. Effect of buildings importance (NRC-1993)

<table>
<thead>
<tr>
<th>E</th>
<th>Building Importance</th>
<th>Design NBC</th>
<th>Low Occupancy N&lt;10</th>
<th>Normal Occupancy N=10 To 300</th>
<th>School, or High occur. N=301 To 3000</th>
<th>Post disaster Very high occur N&gt;3000</th>
<th>Special Operational Requirement</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Pre 70</td>
<td>0.7</td>
<td>1.0</td>
<td>1.5</td>
<td>2.0</td>
<td>3.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Pre 70</td>
<td>0.7</td>
<td>1.0</td>
<td>1.2</td>
<td>1.5</td>
<td>2.0</td>
<td></td>
</tr>
</tbody>
</table>

N= Occupied area x Occupancy density x Duration factor
Primary Use: Assembly 1, Mercantile 0.2, Offices 0.1, Residential 0.05, Storage 0.01, 0.02
Occupancy density Average weekly hours
Average weekly hours
Assembly
5 to 50
50 to 80
50 to 60
100
100
* Duration factor is equal to the average weekly hours of human occupancy divided by 100 not greater than 1.0.

Tables for national Bureau of standards evaluation method

Table (B.4) Grading system of NBS method

Structural systems – Earthquake rating

| Vertical Resisting Elements
<table>
<thead>
<tr>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Type</td>
</tr>
<tr>
<td>---</td>
</tr>
<tr>
<td>GR</td>
</tr>
</tbody>
</table>

Transverse loading

Longitudinal Loading

**Notes:**

1. Symmetry – quantity rating \( SQR = \frac{S + Q}{2} \)

2. Sub-rating \( SR - 1 = \frac{SQR + 2PC}{3} \)

Horizontal – resisting elements

<table>
<thead>
<tr>
<th>Type</th>
<th>Rigidity R</th>
<th>Anchorage &amp; Connections A</th>
<th>Chords (C)</th>
<th>Sub-Rating (SR2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td></td>
<td></td>
<td>Logitudinal</td>
<td>Transtevere</td>
</tr>
<tr>
<td>Floors</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Notes:**

Sub-rating SR2 = Largest of R, A or C

Table (B.4) continued

**Anchorage and connections - ratings**

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Anchorage confirmed – capacity not computed but probably adequate</td>
</tr>
<tr>
<td>2</td>
<td>Anchorage confirmed – capacity not computed but probably inadequate</td>
</tr>
<tr>
<td>3</td>
<td>Anchorage absent</td>
</tr>
</tbody>
</table>
Chords-rating
1  Chords confirmed but capacity not computed
2  Chords unknown but probably present
3  Chords unknown but probably not present
4  Chords absent.

Table (B.4) continued

Exit Corridor And Enclosure Walls Earthquake Rating.

<table>
<thead>
<tr>
<th>Type Of Wall</th>
<th>Reinforcement</th>
<th>Anchorage</th>
<th>Rating Wall</th>
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<tbody>
<tr>
<td></td>
<td>Present</td>
<td>Not Known</td>
<td></td>
</tr>
<tr>
<td>Brick</td>
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<tr>
<td>Brick</td>
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<td></td>
</tr>
<tr>
<td>Concrete Block</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reinforced Conc.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete Block</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Other Life Hazards – Earthquake Rating

<table>
<thead>
<tr>
<th>TYPE OF RISK</th>
<th>RATING</th>
</tr>
</thead>
<tbody>
<tr>
<td>Partitions other than on corridors or stair enclosures</td>
<td>A= Good</td>
</tr>
<tr>
<td>Glass breakage</td>
<td>B= Fair</td>
</tr>
<tr>
<td>Ceiling</td>
<td>C= Poor</td>
</tr>
<tr>
<td>Light fixtures</td>
<td>X = unknown</td>
</tr>
<tr>
<td>Exterior appendages and wall cladding</td>
<td></td>
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</table>

A description of some of the rating for exterior appendages and wall cladding are:

<table>
<thead>
<tr>
<th>Description</th>
<th>Rating</th>
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<tbody>
<tr>
<td>Spacing of anchors appears satisfactory</td>
<td>A</td>
</tr>
<tr>
<td>Size and embodiment of anchors satisfactory</td>
<td>A</td>
</tr>
<tr>
<td>Spacing of anchors appears to be great.</td>
<td>B</td>
</tr>
<tr>
<td>Size and embodiment of anchors appears unsatisfactory</td>
<td>C</td>
</tr>
<tr>
<td>Anchorage unknown</td>
<td>X</td>
</tr>
<tr>
<td>Anchorage corroded or obviously loose</td>
<td>C</td>
</tr>
<tr>
<td>No anchorage</td>
<td>C</td>
</tr>
</tbody>
</table>

ERTHQUAKE GAS CONNECTION

<table>
<thead>
<tr>
<th>PRESENT</th>
<th>NOT PRESENT</th>
<th>NOT KNOWN</th>
</tr>
</thead>
</table>

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Appendix C
Earthquake Resistant Design For Building No "6" Of Skelton Type. According To 1997 UBC
Own weight of slab = 2.5[1×1×.25 – 8×.17×.25×.40] + 8×.014 = .397t / m²
covering material = 0.25m²
Partitions = 0.05 t / m²
Total dead load = .697t / m²
Total story weight = .697×16.4×15.9 = 182.74ton

\[ V = \frac{C_v IW}{RT} \]
\[ I = 1.0 \quad R = 5.5 \quad C_v = 0.18 \]
\[ T = 0.0488(\frac{h_n}{13})^3 \]
\[ T = 0.0488(13)^{3} = .333 \text{ sec} \]
\[ V = \frac{0.18×1×728}{5.5×.333} = 71.54 \text{ ton} \]
\[ C_a = .12 \]

Total shear not. exceed = \( \frac{2.5×.12×1×728}{5.5} = 39.7 \text{ ton} \)

\[ V = 0.11C_a IW \]
\[ V = 0.11×0.12×728 = 9.6 \text{ ton} \]

\[ F_x = \frac{(V - F_i)w_xh_x}{\sum w_ih_i} \]
\[ F_{x=0} \quad T = 0.333 \leq 0.70 \]

\[ F_1 = \frac{71.54×182.74×4}{(182.74×4+182.74×7+182.74×10+182.74×13)} = 8.42 \text{ t} \]
\[ F_2 = \frac{71.54×182.74×7}{6213.16} = 14.72 \text{ ton} \]
\[ F_3 = \frac{71.54×182.74×10}{6213.16} = 21.04 \text{ ton} \]
\[ F_4 = \frac{71.54×182.74×13}{6213.16} = 27.35 \text{ ton} \]

<table>
<thead>
<tr>
<th>level</th>
<th>W (ton)</th>
<th>H (meter)</th>
<th>( w_xh_x )</th>
<th>( C_{VX} )</th>
<th>( F_x )</th>
<th>( V_x )</th>
<th>( M_X )</th>
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<tr>
<td>4</td>
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<td>3</td>
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<td>10</td>
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<td>7</td>
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<td></td>
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<td></td>
<td>( \Sigma \ 6213.16 )</td>
<td></td>
<td></td>
<td>( \Sigma \ 702.59 )</td>
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<tr>
<td>Col #</td>
<td>b</td>
<td>h</td>
<td>A</td>
<td>I_x</td>
<td>I_y</td>
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<td>.15</td>
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<td>.0247</td>
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<tr>
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<td>.70</td>
<td>.175</td>
<td>.007</td>
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<td>.045</td>
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<td>.175</td>
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<td>.071</td>
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<td>.15</td>
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<td>.125</td>
<td>.003</td>
<td>13.9</td>
<td>.0782</td>
<td>4.7</td>
</tr>
</tbody>
</table>

\[
y = \frac{\sum y_{i,j}}{\sum I_y} = \frac{0.584}{0.063} = 9.53
\]

**Torsion caused by eccentricity** = \( F_x \times 2.53 \)

Addition eccentricity = 5% of each direction = \( \frac{5}{100} \times 14 = 0.7 \ m \)

\[
x = \frac{\sum I_x x_i}{\sum I_x} = \frac{0.239}{0.0277} = 8.60 \ m
\]

\( x_c = 1.35 \ m \)

**Torsion** = \( F_y \times 1.35 \)

\( 0.725V_y \), Addition torsion = 0

\[
\overline{F_{ix}} = F_{ix} + \overline{F_{ix}}
\]

\[
\overline{F_{ix}} = \frac{\sum I_{iy} \times F_x}{0.007} = 0.007 F_x = 0.12 F_x
\]

\[
\overline{F_{ix}} = \frac{\sum (C_{yi} \times I_{iy}) \times V_y}{1.1} = 4.70 \times 0.007 (2.53 F_x + 0.70 V_y) = 0.048 F_x + 0.0134 V_y
\]

**Total force of each level** = 0.168\( F_y \) + 0.0134\( V_y \)

\( F_4 = 4.96 \text{ ton} \) \( F_3 = 4.18 \text{ ton} \) \( F_2 = 3.32 \text{ ton} \) \( F_1 = 2.37 \text{ ton} \)

<table>
<thead>
<tr>
<th>Calculation of moment (t.m)</th>
<th>Calculation of shear (Ton)</th>
<th>Loads (Ton)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( M_4 = 82.05 )</td>
<td>( F_4 = 4.96 )</td>
<td>4.18</td>
</tr>
<tr>
<td>( M_3 = 227.22 )</td>
<td>( F_3 = 9.14 )</td>
<td>4.96</td>
</tr>
<tr>
<td>( M_2 = 416.50 )</td>
<td>( F_2 = 12.46 )</td>
<td>3.32</td>
</tr>
<tr>
<td>( M_1 = 702.59 )</td>
<td>( F_1 = 14.83 )</td>
<td>2.37</td>
</tr>
</tbody>
</table>
Design for shear

\[ V_{n,\text{max}} = 2.65 \sqrt{f_c \cdot h d} \]

\[ = 2.65 \sqrt{300 \times \frac{25 \times 80 \times 70}{1000}} = 64.25 \text{ ton} \]

\[ V_{u,\text{max}} = 0.85 \times 64.25 = 54.6 \text{ ton} \]

\[ V_{c} = 0.53 \sqrt{f_c \cdot h d} \]

\[ = 0.53 \sqrt{300 \times \frac{25 \times 80 \times 70}{1000}} = 12.85 \text{ ton} \]

\[ \Phi V_{c} = 0.85 \times 12.85 = 10.92 \text{ ton} \]

\[ \Phi \frac{V_{c}}{2} = 5.50 \text{ ton} \]

For zone (1)

Horizontal shear reinforcement

\[ \rho_h = 0.0025 \]

\[ S_1 = \text{the smaller of} \begin{cases} 61.2 \text{ cm} < S_{1,\text{max}} = \frac{l_w}{5} = 14 \text{ cm} \\ 3h = 75 \text{ cm} \\ 45 \text{ cm} \end{cases} \]

\[ S_{2,\text{max}} = 14 \text{ cm} \]

\[ \frac{A_v}{S_1} = 0.0025h = 0.0625 \text{ cm}^2/cm \]

Taking two stirrups 8mm

\[ \frac{2 \times 0.5026}{S_2} = 0.0625 \]

\[ S_2 = 16 \text{ cm} \quad \text{not o.k} \]

Vertical shear reinforcement

\[ S_1 = \text{the smaller of} \begin{cases} 23.30 \text{ cm} \\ 3 \times 25 = 75 \text{ cm} \\ 45 \text{ cm} \end{cases} \]

\[ S_1 = 23.30 \]

\[ \frac{A_v}{S_1} = 0.0025h = 0.0025 \times 25 = 0.0625 = \frac{2 \times 1.53}{S_1} \]

\[ S_1 = 61.20 < S_{1,\text{max}} \]

use Φ14mm bars @ 9 cm
Design for flexure:

\[ M_u = \phi \left[ 0.50A_{sf}f_y I_w \left( 1 + \frac{P_u}{A_{sf}f_y} \left[ 1 - \frac{C}{I_w} \right] \right) \right] \]

\[ C = \frac{\omega + \alpha}{I_w} = \frac{2\omega + 0.85\beta}{2\omega + 0.85\beta} \]

\[ \omega = \frac{A_{sf}f_y}{L_w hf_c \ldots and \quad \frac{P_u}{L_w hf_c} = \frac{P_u}{2.191.50} = 0.991.1.50} \]

For \( \phi 14 \text{mm @ 9cm bars} \)

\[ A_s = 1.54 \times \frac{2 \times 70}{9} = 23.95 \text{ cm}^2 \]

\[ \omega = \frac{23.95 \times 4200}{70 \times 25 \times 300} = 0.192 \]

\[ \beta = 0.85 \times \frac{0.05}{70} (300 - 280) = 0.836 \]

\[ \alpha = \frac{P_u \times 1000}{70 \times 25 \times 300} = 0.0019P_u \]

\[ C = \frac{0.1916 + 0.0099P_u}{2 \times 1.91 + 0.85 \times 0.836} = \frac{0.192 + 0.0019P_u}{1.093} \]

For zone 1:

\[ P_u = 0.99D + 1.1E_v = 0.99D + 1.1 \times 0.5C ID \]

\[ P_u = 0.99 \times 0.25 \times 0.25 \times 0.25 \times 0.12 \times 1 \times 0.5 \times 0.25 \times 0.25 \]

\[ P_u = 1.386 \text{ ton} \]

\[ C = \frac{0.192 + 0.0019 \times 1.386}{1.093} = 0.178 \]

\[ M_u = 0.90 \left[ 0.5 \times 23.95 \times 4200 \times 0.70 \left[ 1 + \frac{1.386 \times 1000}{23.95 \times 4200} \right] \right] \left( 1 - 0.178 \right) \]

\[ M_u = 264 \text{ t.m} > 82.05 \text{ o.k} \]