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Analytical Study of Model Rubberized Concrete Base Isolation System for Buildings in developing Countries

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Abstract: *Materials which improve damping and ductility during structural response to earthquakes are very beneficial. Concrete incorporating scrap rubber is one of them. This study analysed the dynamic response of a 3-bay,4-storey reinforced concrete frame designed according to Eurocode 8 and the response of the same frame on an FRP-confined rubberized concrete foundation. Using confined rubberized concrete as short foundation columns between the ground and the frame lowered shear demand on a structure by 70%, and acceleration demand was reduced by 75 %. The reduction in seismic demand was mainly due to deflection of the earthquake force by the deformable foundation system in a manner analogous to base isolation systems. The base-isolated frame performed efficiently with an average of over 60% reduction in interstorey drifts. The implication of this is that a less expensive frame designed according to the provisions of Eurocode-2 can perform satisfactorily if founded on FRP-confined rubberized concrete columns foundation system. The reduction in force demand on the structure implies lesser material for the superstructure which implies lesser cost of shelter. The proposed deformable system from FRP-confined rubberized concrete is not only cheap, simple to construct with a little technical know-how but is also environmentally friendly. With the current challenges in providing good quality shelter in earthquake-prone, developing countries, providing concrete which is not only cheaper but with enhanced dynamic properties, will go a long way to provide safe structures for humanity. Apart from this, recycling scrap tyre rubber to be used as aggregates in the production of concrete for structural applications will in no small measure result in a cleaner and sustainable environment.*

Keywords: *Base Isolation, Dynamic response, Earthquake, FRP, Developing Countries, Rubberized, Seismic*

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I. Introduction

The next most basic need of man after food is shelter. Buildings in developing countries are provided by the citizens themselves who erect structures that they can afford. These buildings most times fail quality control requirements and are poor in strength. Due to the low strength and non-ductile nature of the buildings, they collapse at the slightest ground movement and eventually, lives are lost. Designing a structure to be ductile in its response to enormous forces generated during earthquakes cannot be overly emphasized. There is the need to design for damage. In other words, post-elastic behaviour should be allowed in structural response to some severe reference earthquake. The advantages of ductile design include ductile response of the structure which in turn implies energy dissipation, damping of structural response, reduced seismic forces, safety against collapse, and economy in design. For the impoverished citizens of most developing countries, the provision of this shelter must be done in the most economical way.

Statistics provided by [1] indicate that even though the field of earthquake engineering has developed considerably in the last few decades, fatalities arising from earthquake events have been on the increase especially in developing countries. He presented a graph comparing a history of earthquake fatalities and world population growth history. This comparison is shown below in Fig. 1.1. Most recent examples include Haiti and Nepal. Haiti lost about 200,000 lives from a 7.0 magnitude earthquake in 2010 while a more developed Japan lost 20,000 lives after a massive 9.0 magnitude earthquake the following year, mainly from the resulting tsunami caused by the earthquake and not the earthquake itself. The relatively less populated Nepal in 2015 recorded over 9000 deaths from the 7.9 magnitude Ghorkha earthquake. It, therefore, follows as [2] puts it that two basic criteria govern structural response to earthquake ground motion; the intensity of shaking and the quality of the buildings. Another factor influencing seismic risk is population density. The poor quality of buildings and high population density has been responsible to a greater extent for the numerous casualties during extreme events.

The research of [3] opined that the world population is projected to double in about 50 years which means that about 1 billion additional housing units will be needed to meet the demand not forgetting the fact that inadequate housing has been a major problem in developing countries.

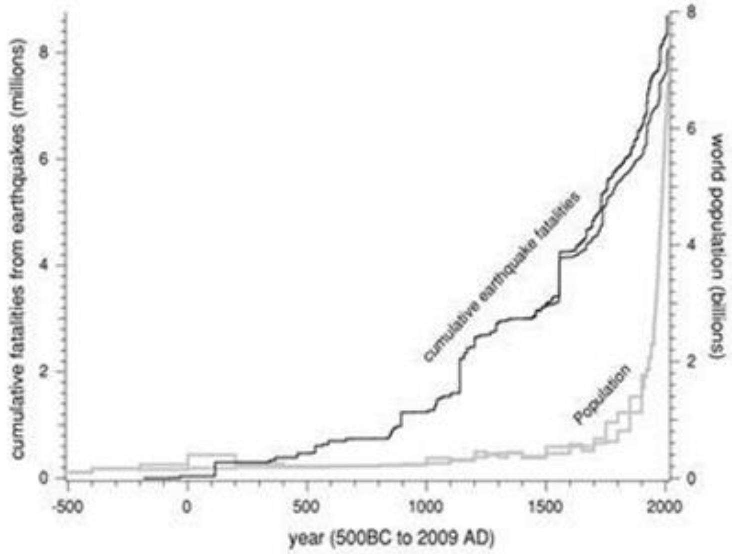

Fig 1.1: *Comparison of Earthquake Fatalities with World Population Growth* [1]

About 68% of the 79 most populated cities in seismically active regions of the world are located in developing countries. For most of these countries, social instability makes it impossible for the state building regulations to enforce the stringent requirements needed for proper seismic design and provision of housing in its municipalities. The poor quality of buildings in such regions, exacerbate the level of casualty during an earthquake as the buildings fall directly upon the occupants.

The need for a cheap alternative method to achieving structural ductility, simpler design procedure, cheap but efficient construction materials, and simple construction techniques for these developing areas cannot be overemphasized. The requirement for ductile structural response against seismic forces has led to the developments of special provisions for seismic analysis, design, material selection and methods of construction. These provisions recommend the use of certain materials like elastomeric bearings to isolate the structure from the ground, deformable aggregates for concrete, adherence to special reinforcement detailing and confinement for concrete among other attempts at improving ductility. A highly deformable aggregate that has drawn a lot of research interest over the past two decades is scrap rubber from waste vehicle tyres.

Scrap rubber is cheap, readily available, chemically inert, non-biodegradable and constitutes a menace to the environment. In fact, in some countries, it has been banned from disposal in landfills [4]. However, research has shown that it can be processed into aggregates and incorporated in concrete. Concrete incorporating rubber, although is characterized by low compressive strength can be a cheap and more ductile alternative than conventional concrete. When compared with conventional concrete, rubberized concrete is more deformable under pre-failure loads, is tough, it has good impact resistance, low shrinkage rate, and exhibits better crack-resistance [5]. It is also chemically inert, possesses higher damping and displays improved curvature ductility. This implies good energy dissipation and makes it a promising type of concrete for seismic regions [3,6].

Compressive strength loss usually associated with concrete incorporating rubber can be negated by introducing lateral confinement to the concrete [7]. A popular material used for confinement is fibre reinforced polymers (FRP). Confinement in this context can be described as providing restraints around an element in order to prevent or delay lateral dilation under load, thereby increasing the capacity of such elements. The presence of rubber in concrete results in a deformable concrete which when confined increases compressive strength. This combined behaviour of enhanced strength, improved deformability and ductility is a desired structural behaviour in earthquake-resistant design.Consequently, the suitability of rubberized concrete as base-isolator for buildings is investigated in this research.

1.1 Aim and Objectives

The aim of this research was to analyse the suitability of short FRP-confined rubberized concrete columns as a base isolator for buildings.

This was achieved through the following objectives:

- Design and Analysis of an earthquake-resistant 3-bay 4-storey reinforced concrete frame
- Analysis of the earthquake-resistant frame separated from the foundation base by short FRP-confined rubberized concrete columns,
- Assessing the dynamic response of both frames when subjected to varying peak ground accelerations of a reference earthquake.

These was carried out with the aid of available experimental data on the stress-strain relationship of 3-layers FRP-confined rubberized concrete as reported in[3].Structural modeling and performance werecarried through simulation using Drain-2dX software.

1.2 Significance of the Research

Developing countries are plagued with problems like poverty, civil unrest and corruption which make it almost impossible for the provision of decent shelter for citizens. The citizens in desperation are left with no choice than to provide shelter for themselves using substandard building materials and naive construction practices. Most traditional building materials like stones, bricks, cement mortar and concrete are inherently brittle and make buildings to fail in a sudden brittle manner when subjected to the force demands of earthquake ground vibration. Rubberized concrete is an economical and promising concrete for specialized applications such as is required for earthquake resistant structures. Abundant studies already abound concerning incorporating scrap tyre in concrete. Recently, there have been more studies on the suitability of this type of concrete for dissipative zones such as beam-column connections. Results from several studies have been reported on its attendant effects, advantages and disadvantages. Recommendations have also been made on the likely application of this type of concrete. However, no attempt has yet been reported on actual analysis assessing its suitability for base isolation of buildings. Most studies so far have been concentrated on structural elements and joints. Research on using deformable rubberized reinforced concrete elements for foundations may reveal new approaches to the seismic design of structures. Confined rubberized concrete can provide ductile foundation systems without compromising strength requirements. Such deformable foundation will provide a cheap means of achieving improved damping and energy dissipation. Increased energy dissipation at the foundation could also reduce the base shear during lateral excitation. With the recent developments in FRP confinement for columns[8], it has been shown that strength loss (as a result of the presence of rubber particles in concrete) can be eliminated. In fact, further strength gains were reported. The result of FRP confinement has been a more ductile and deformable rubberized concrete suitable for structural application in seismically active zones. Conventional seismic design and provision of structures are expensive for most residents of earthquake-prone regions. The solutions from this research shall provide a simple, innovative and affordable foundation system for use in developing countries prone to seismic activities. The use of unwanted waste material like used tyres which has continually become a menace to the environment is also a good practice.

II. Methods and Analyses

For a reference magnitude of ground motion, a structure needs to be designed in such a way that its capacity balances the force demand incident upon it by the ground [9].For conventional structural analysis and design, the materials are truly elastic to the magnitude of applied static forces. However, for a dynamic load such as earthquake ground vibration, the response of the structure imposes excessive forces on the material and the elastic limit is exceeded. It, therefore, becomes necessary to consider and appropriately idealize this post elastic behaviour. Fig 2.1 illustrates the concept of idealizing ductility. The ratio of the expected peak displacement to the yield displacement is referred to as ductility (Δ_u/ Δ_y).

2.1 Basic Principles of Ductility

Fundamental principles of seismic design of buildings require that the design be simple with clearly defined load paths for transmitting the seismic forces incident on the structure. The benefit of this is that the structural response can easily be predicted. A structure which is symmetric in both geometry and lateral resistance performs better than one which is asymmetric in either or both characteristics. Asymmetry creates stressconcentration and likelihood of parts of the structure being overloaded during seismic excitation. EN1998-1:2004 defines the criteria for structural regularity. A structural system with a higher degree of redundancy will also perform better than a less redundant one. The consequence of increased redundancy is increase in energy dissipation and reduction of seismic demand on the structure. EN1998-1:2004 also provides that at each storeylevel, the slab be designed as a diaphragm so that seismic effects are evenly distributed to all vertical elements. Very large openings in floor slabs, as well as structural systems which are very long in plan, should be avoided. If unavoidable, then the different parts of the building should be separated. The foundation must also be designed in such a way that it provides uniform lateral excitation to all the parts of the structure [10]. EN1998-1:2004 also categorizes seismic design of reinforced concrete structures into 3 classes of, Low Ductility (DCL), Medium Ductility (DCM) and High Ductility (DCH). As the names imply, the High ductility class has the highest energy dissipation capability. DCL structures are designed to meet the requirements of Eurocode 2 and not Eurocode 8 since they are meant for regions with very low seismic activity. These classes of buildings can belong to any of the following structural systems of (i) frame, (ii) ductile wall, (iii) dual, (iv) large lightly reinforced walls, (v) inverted pendulum and (vi) torsionally flexible systems. A detailed description of these structural systems is given in EN1998-1:2004.

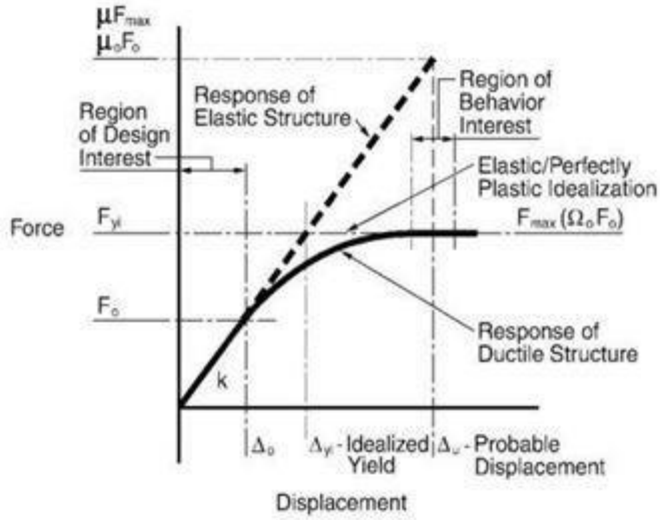

Fig 2.1: *Force-displacement relationship of a ductile structure [10]*

2.2 Base Isolation

In base isolation systems, highly deformable elements are introduced in the base of a structure to improve structural ductility and enhance dynamic response to earthquake ground motion. The use of these deformable elements to de-couple the superstructure from the direct seismic action is referred to as base isolation. The most widely used highly deformable elements are the low-stiffness rubber elements. They are composed of elastomeric bearings which are made up of steel–reinforced natural rubber or Neoprene [12]. These bearings which are sandwiched between the foundation and the superstructure have low horizontal stiffness and hence deflect a good amount of the seismic forces from the ground during earthquakes. The force demand on the superstructure is consequently reduced with resulting mild structural response and damage. Erik *et al* [12] reported that the seismic design codes for base isolation of structures have recommendations which are very conservative and as such results in the provision of large elements as isolators. The usual alternative to reducing the size of these large elements (which result in loss of required space) is to supplement the lateral resisting system with other damping devices. This alternative may reduce the base drift but then increase interstorey drifts and floor acceleration. There is also the problem of designing these systems for a maximum earthquake event with a long return period. This makes the isolation system too stiff and thereby increases the force demand on the superstructure for the more frequent low magnitude earthquake events [12]. A cheap alternative material with huge potential in base isolation systems is FRP-confined rubberized concrete.

2.3 FRP-Confined Rubberized Concrete

Materials which improve damping during structural response to earthquakes are very beneficial. Concrete incorporating scrap rubber from waste tires has attracted a good number of research interests over the last two decades. These researches agree that incorporating rubber in concrete improves its properties in terms of toughness, impact resistance, damping, ductility, lightweight and durability but with reduced compressive strength and elastic modulus [5-7,13-24].

The disadvantage of lowering compressive strength due to the inclusion of rubber can be effectively eliminated by lateral confinement using FRP. In fact, strength increases have been observed with FRP confinement with increased rubber content [7,25,26]. Increasing the rate of cyclic loading for FRP-confined rubberized concrete results in increased compressive strength and ductility for cylindrical specimens [26]. Using rubberized concrete for structural elements like bridge columns, beams, columns and foundation for building frames in seismic zones due to ductility requirement are feasible [7]. The capacity of a strength-deficient reinforced concrete frame was enhanced by FRP fabric wrapping (impregnated with epoxy resin), around the joints of the frame. At the end of a shake-table test, global damage was reported to be 65% less than the damage experienced before retrofitting [27].

The next method worthy of note which has been found to substantially increase ductility and energy-dissipation of reinforced concrete frame elements is confinement through the use of post-tensioned metal straps (PTMS). Garcia *et al* [28] prescribed PTMS for seismic strengthening of beam-column joints. A single bay 2-storey full scale building was tested using a shake table. The bare building could only resist a peak ground acceleration intensity of 0.15g. The test was stopped, and the frame was repaired by a method of epoxy-injection to seal up the cracks. PTMS were then used to strengthen the building at the columns and joints, after which the building was able to go through 0.35g peak ground acceleration with minimal damage.

The problem of strength loss seems to be the only factor discouraging the use of this type of concrete for structural applications therefore ways have been sought to counter this strength reduction. Several researches including the works of [3,20,26,29,31], have postulated different methods of attempting to negate the strength loss which characterizes rubberized concrete, however, only FRP-confinement of rubberized concrete elements as described in [7,8,27], seem more viable and promising and hence have been chosen for the analytical models used in this study.

III. Reinforced Concrete Models

Base isolation for structures employs a deformable system to create a "soft storey" at the base of the building such that the superstructure is de-coupled from the direct impact of the seismic force. Similarly, introducing some deformability and damping by the use of rubberized concrete at the foundation level will reduce the force demand on the global frame. In this research, an attempt is made to test a reinforced concrete frame model on a deformable foundation system under simulated earthquake. The structural responses of the models were compared with that of a conventional reinforced concrete frame (referred to as "control frame"). The earthquake time history was scaled at 5 different peak ground accelerations (i.e, 0.28g, 0.30g, 0.34g, 0.40g and 0.44g). The elements of these proposed foundation models were assigned the properties of cylindrical concrete specimens prepared by replacing 60% coarse and fine aggregates with crumb rubber from waste vehicle tyres and confined with 3-layers of Aramid FRP. The geometry of the deformable system also provides increased redundancy for the frame. The arrangement of the deformable foundation should provide more load paths into the ground as against the single load path provided by a conventional pad foundation. Fig 3.2(a) and Fig 3.2(b) shows the Control frame and the Base-isolated frame respectively.

Experimental results on the stress-strain properties of 150 x 300 mm cylindrical rubberized concrete specimens confined with three (3) Layers of Aramid FRP were obtained from [3]. Fig 3.1 shows an idealised stress-strain relationship of rubberized concrete obtained by partially replacing fine and coarse aggregates by 60% (by volume) with crumb rubber obtained from used vehicle tyres

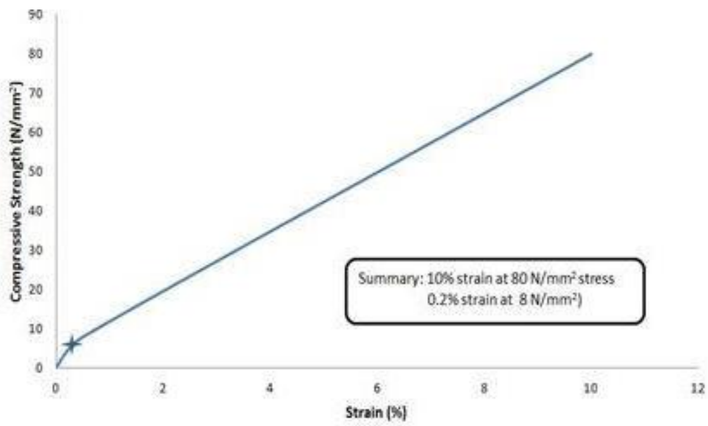

Fig 3.1: *Stress-strain properties of FRP-confinedrubberised concrete (60% crumb rubber) confined with 3layers of Aramid Fabric [3]*

3.1.0 Frame Models

3.1.1 Control Frame

This Frame used as the control for this research is a 3-bay, 4-storey reinforced concrete frame designed according to the requirements of EN1998-1:2004. See Fig 3.2(a) for frame dimensions. The chosen frame size is representative of both residential and office buildings in developing countries with seismic activities. All dimensions on the figures are in millimeters.

3.1.2 Base Isolation Model

This is the control frame mounted on short circular FRP-confined rubberized concrete columns with dimensions as shown in Fig 3.2. The ends at the base of the FRP-confined rubberised concrete column deformable system have been modelled as pinned joints. The implication of this is a reduction in stiffness at the base of the frame and this is sought to deflect the direct earthquake force from the structure, thereby achieving a base isolation system. The stress-strain curve of the FRP-confined rubberized concrete is shown in Fig 3.1 and the properties of the constituting materials of the deformable foundation system are shown as follows:

- Compressive strength of FRP-confined rubberized concrete, $f_c = 80$ N/mm^2
- Maximum strain of FRP-confined rubberised concrete, $\varepsilon_{cc} = 0.100$
- Modulus of Elasticity of the above concrete E = 400 N/mm^2

- Ratio of final stiffness to initial stiffness of the above concrete k_2/k_1 = 20%

These properties were obtained from stress-strain experiments of 150mm x 300mm cylindrical rubberized concrete elements confined with 3 layers of aramid fibres by [3]. For the structural design of these models, the following steel properties were used:

- Steel reinforcement f_y = 250 N/mm^2
- Steel yield strain f_y= 0.00109

The design for both the control model and the base-isolation model included the following steps:

- Definition of structure and geometry,
- Preliminary sizing of elements,
- Definition of element properties (Area, Length, Modulus of Elasticity, Moment of Inertia),
- Discretization of the structure into elements (i.e. beams and columns),
- Definition of element connection types and connectivity
- Definition of gravity and seismic actions,
- Selection of soil type,
- Calculation of the EN1998-1:2004 structure-specific behaviour factor (*q-factor*),
- Application of *q-factor* to the reference Elastic Response Spectrum,
- Running the static linear elastic response analysis,
- Design of sections,
- Selection of earthquake time history,
- Running the non-linear time history dynamic analysis,
- Assessment of non-linear response of frame.

3.2 Design Procedure

The structure used for the investigations of this research is a 3-bay, 4-storey reinforced concrete frame modeled and analysed in Drain-2Dx and designed according to the provisions of EN1998-1:2004. The study seeks to concentrate the force demand at the base of the structure by investigating a proposed confined rubberized concrete deformable foundation system for conventional moment resisting frames. The proposed foundation system seeks to increase energy dissipation and the decoupling of the superstructure from the direct earthquake force, thereby reducing the force demand on the frame.

The standard method for computing seismic effects is linear-elastic, multimodal response analysis using the design response spectrum (EN1998-1:2004-3.2.2.1). When the behaviour factor is used to reduce the elastic response spectrum, the resulting spectrum is known as the design spectrum. The Eurocode-8 method is to analyse and design the structural system using elastic linear analysis for a reduced seismic force (design response spectrum). This implies that the structure will undergo post-elastic deformation when hit by the actual ground motion. The non-linear deformation of the structure is accounted for by this behaviour factor *q*, defined in the design code. The behaviour factor often referred to as the *q*-factor is given by;

$$q = q_o k_w \quad (3.1)$$

where q is the behaviour factor, q_o is the basic value of the behaviour factor and $k_w$is a factor that describes the predominating mode of failure of structural systems with walls. The control structure is designed according to the requirements of EC 8 for medium ductility reinforced concrete frames (DCM). The frame was analysed using linear elastic analysis for a behaviour factor-reduced seismic force. The behaviour factor for a DCM multi-storey multi-bay structure is given as:

$$q = 3 \times 1.3 = 3.9$$

EN1998-1:2004 provides the following equations for reducing the elastic response spectrum by the behaviour factor q:

$$0 \le T \le T_B : S_d(T) = a_g.S.\{\frac{2}{3} + \frac{T}{T_B}.(\frac{2.5}{q} - \frac{2}{3})\} \quad (3.2)$$

$$T_B \le T \le T_C : S_d(T) = a_g.S.\frac{2.5}{q} \quad (3.3)$$

$$T_B \le T \le T_C : S_d(T) \{ =a_g.S.\frac{2.5}{q}.[\frac{T_C}{T}]\} \ \ or \ \ \{ \ge \beta.a_g \quad (3.4)$$

$$T_D \le T : S_d(T) \{ =a_g.S.\frac{2.5}{q}.[\frac{T_C T_D}{T^2}]\} \ \ or \ \ \{ \ge \beta.a_g \quad (3.5),$$

where; *$S_e(T)$ is the elastic response spectrum*
T is the vibration period of a linear single degree of freedom system,
$a_g$is the design ground acceleration on ground type A,
T_B is the lower limit of the of the period of the flat peak acceleration plateau,

T_C is the upper limit of the flat peak acceleration plateau,
T_D is the value defining the beginning of the constant displacement response range of the spectrum,
S is the soil factor,
η is the damping correction factor with a reference value of 1.0 for 5% viscous damping
{source: EN1998-1:2004. 3.2.2.2-1(P)}

This process of reducing the elastic response spectrum to the design spectrum was done by using a Matlab script to solve for the above equations. The design spectrum file was executed alongside Drain-2dX. The Design Response Spectrum is shown in Figure 3.3. Linear elastic analysis of the frames was performed for 0.40g peak ground acceleration (elastic response spectrum). Reinforcement areas were provided, and the resistances assigned to the elements in the input file, then non-linear time-history dynamic response was accessed for peak ground accelerations of 0.28g, 0.30g, 0.34g, 0.40g, and 0.44g for both models.
Several analytical tools can be used for the process of design and investigation among which DRAIN-2Dx finite element software package has been chosen for this research.

3.2.1 Design of Control Frame

A 2D model of a 4-storey, 3-bay multi-storey frame structure was used as the control structure. Drain-2dX uses an input text file (*drain.inp*) which requires that the structure be discretized into elements joined at nodes with nodes and elements appropriately numbered. For the purpose of consistency of units, quantities were defined in kilonewtons and centimetres. Usually, at the start of design, a number of different element cross-sections commensurate with load demands are proposed to be used for robust and economic design. However, for the purpose of simplicity, a single cross-section shall be defined for the column group and another cross-section for the beam group with their respective geometric properties included. Properties calculated and needed as inputs include, cross-sectional areas, moment of inertia and shear area of the elements. The elements were linked to the nodes and the respective properties of the elements were defined in the *drain.inp* file. Gravity loads were determined and converted to nodal loads to be used in gravity analysis. The loads at the nodes were then converted into nodal masses by dividing the load at each node by acceleration due to gravity of 981cm/s^2. While the nodal loads are required for gravity analysis, the nodal masses are used by the program for dynamic analysis. The loads and the masses were then assigned to the nodes in the file *drain.inp.* A design spectrum was generated with the aid of a Matlab script. The inputs include a file with *T-S_d* values, where T is the period in seconds and S_d is the corresponding design spectrum amplitude given as a proportion of *g*. A peak ground acceleration of 0.40g was used on ground *type B*. The Frame was designed for medium ductility class according to the provisions of EN1998-1:2004.

3.2.1.1 Structure, Geometry and Analysis

Both frame models were discretized and entered into the analysis software as coordinates with the material properties and gravity loadings. Linear elastic response analysis was run, and it returned two results amongst other outputs. The first was a table of Moments, Shear Forces and Axial Forces for each element due to gravity load combination (∑Gk + ∑Ψ2Qk); and a second part of the result which showed a table of values for moments, shear forces and axial forces for each element as a result of the earthquake excitation. The program superposes the four modes of vibration response of the structure in order to arrive at the force demand due to earthquake ground motion. The method of superposition used is the Square Root of the Sum of the Squares SRSS. This calculated demand on the structure is referred to as the design effect $E_{D,s}$due to seismic action A_{Ed}. Figure 3.11 below shows the discretised model of the frame into elements of beams and columns with numbered nodes to connect them at the joints. Element Dimensions (frame dimensions are in mm).

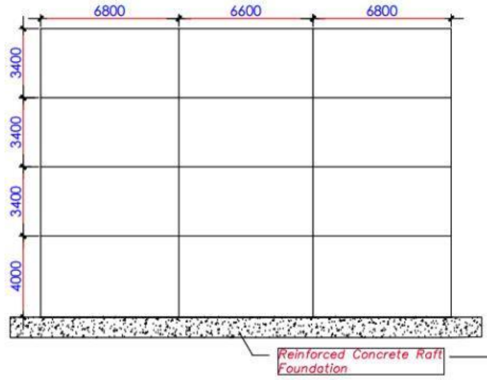

***Fig 3.2a**: Earthquake Resistant Reinforced Concrete Frame designed to Eurocode 8 (Control Frame)*

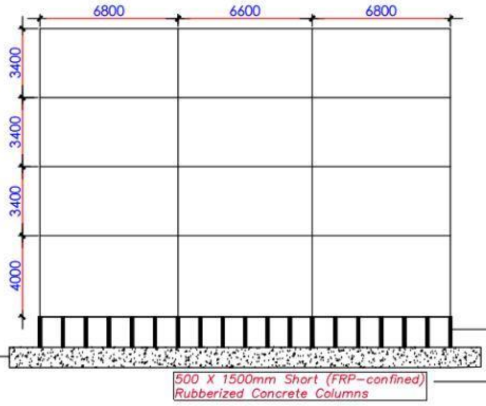

***Fig3.2b**: Earthquake Resistant Reinforced Concrete Frame on FRP-Confined Rubberized Concrete (Base-isolated Model*

1. Beam web = 400 x 300 mm
2. Column = 500 x 500 mm

Loads

1.Spans: $l_{x1} = l_{x2} = 6.8$ m; $l_{x2}= 6.6$ m; $l_y = 7.0$ m
2.Storey height: *Ground storey*: h = 4.0 m;
Other storeys: h = 3.4 m.
3.Floor slab thickness: $h_f= 0.2$ m;
4.Unit weight of Concrete, $r_c = 25$ kN/m^3
5.Variable Load: *Top floor:*$q_k= 1$ kN/m^2
Other floors: $q_k = 2.5\ kN/m^2$

The seismic action is represented by design spectrum Type 1. The Ground type and Peak ground acceleration is given below:

1.Ground type: A
2.Peak Ground Acceleration $a_g= 0.40$ g
3.Type of Concrete Building =Multi-storey, multi-bay frame

The building is designed for ductility class medium: DCM

3.2.1.2 Analysis 1: Linear Elastic Response Spectrum Analysis

The frame was analysed using elastic linear analysis for a design seismic force as defined by EN1998-1:2004. For a DCM multi-storey single-bay frame concrete building as this, the behaviour factor q is given as:

$$q = q_o k_w$$

where$q$ is the behaviour factor, q_o is the basic value of the behaviour factor and k_w is a factor that reflects the prevailing failure mode in structures with walls. For pure frames, $k_w = 1.0$. Hence q = q_o and:

$$q_o = 3\alpha_u/\alpha_1$$

where; $\alpha_u/\alpha_1 = 1.3$, is the over-strength ratio for a regular multi-storey multi-bay frame concrete building. Therefore, the behaviour factor by which the seismic force is reduced by is:

$$q = 3 \times 1.2 = 3.9$$

Fig. 3.3 shows the Design Spectrum used in the linear elastic analysis of the frames.
The linear elastic analysis was performed for a 0.40g peak ground acceleration earthquake on soil type B. This soil of type is described by EN1998-1:2004 as “very dense sand, gravel, very stiff clay”, h > 30m” with velocity of shear waves in this soil as between 360-800 m/s. Results from the linear elastic analysis gave the mode shapes and periods of 4 modes. The force demands on the nodes of the frame are made up of two parts:

1. Moments, shear forces and axial forces due to gravity loads
2. Moments, shear forces and axial forces due to seismic excitation

Since the ground motion goes back and forth, it is appropriate to represent the seismic demands under two cases; case 1 for left-to-right action and case 2 for right-to-left action.
The adequate reinforcement areas were provided for the beam and column element sections as required by the provisions of EN1998-1:2004. The capacities of the respective element groups were fed into the *drain.inp* file as the resistances of the elements. Non-linear time-history dynamic analysis was then executed by subjecting the frame to five differently scaled peak ground accelerations for the earthquake time history used. The non-linear response of the frame was then assessed in terms of interstorey drifts and plastic hinge formation. For lack of space, details of the design are not shown in this report.

3.2.2 Design of Base Isolation Model

The combined force demands on the floor columns of the control frame were used to propose the element sizes and cross-sections of the deformable foundation system of the control. The philosophy behind the adopted geometries for these deformable foundation systems has been explained earlier in sections 3.1.2-3.1.5. Following the same design procedure as that of the control frame, the resistances of the FRP-confined rubberized base-isolator columns were designed and the non-linear response of the models was assessed following the results of the non-linear time history dynamic analysis for 5 peak ground accelerations of the simulated earthquake. The structural responses of both control frame and base-isolated frame are discussed in section IV.

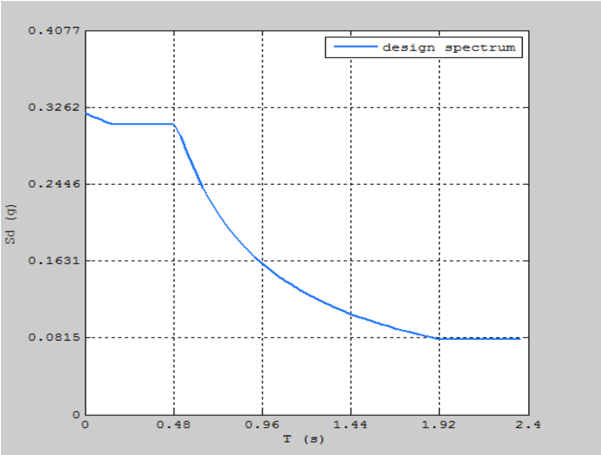

Fig 3.3: *Reduced (Design) elastic spectrum for elastic analysis of the frames*

IV. Results and Discussion

4.1 Linear Elastic Response Spectrum Analysis

For each frame model, a linear elastic multi-modal response analysis was carried out to determine the modal properties and force demand on the different elements of the structure. It was from the results of this analysis that the capacity design of the various beam and column sections was carried out. Figure 4.1(a and b), shows the mode shapes of the control frame and FRP-confined rubberized concrete frame (Base-isolated frame). See Fig 4.1 for mode shapes of the Control frame and Base-isolated frame.

The Base-isolated frame has 5 mode shapes because of an extra degree of freedom introduced by the deformable foundation system.

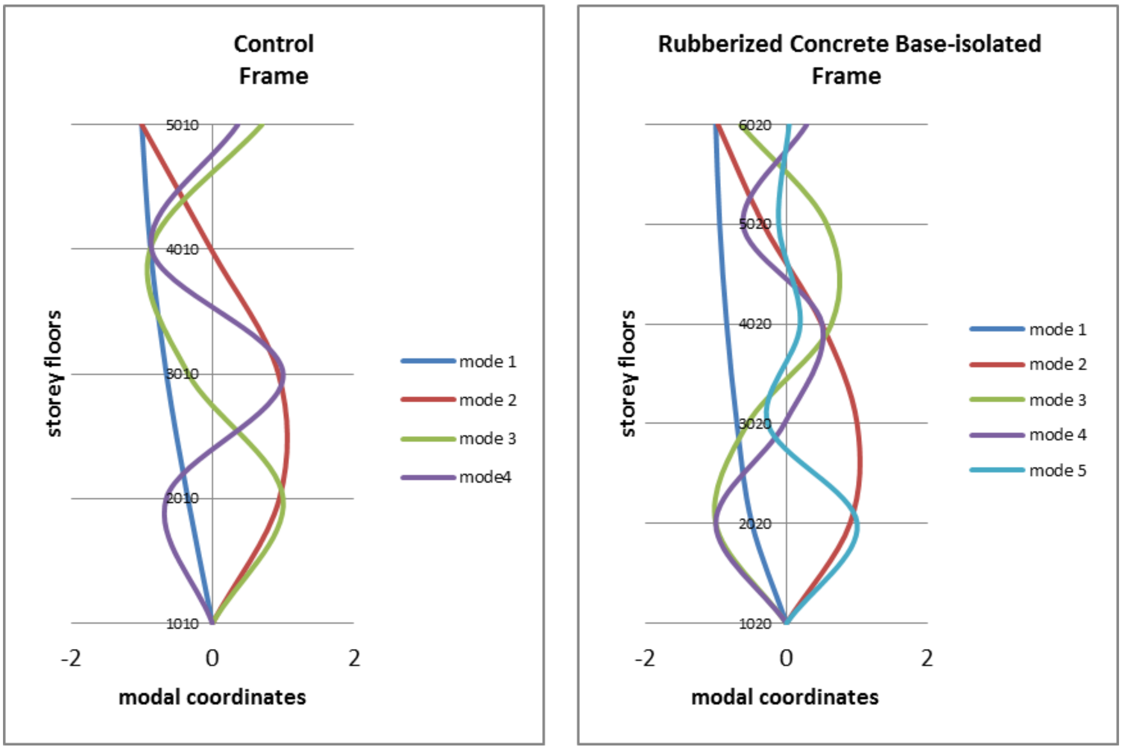

Fig4.1: *Mode Shapes of Control Frame and Base-Isolated Frame*

Table 4.1 also shows the periods and effective modal masses of the frame models. The incorporation of a deformable concrete system at the foundation improved the ductility of the structure. Increase in its period of vibration implies that the structure is pushed to the right side of the earthquake spectrum. The more to the right side of the spectrum the structure's period is pushed, the lesser the acceleration of the structure and consequently

a reduction of the force demand during excitation by the reference earthquake. The base-isolated model has a longer period.
Table 4.1 below also shows the respective periods and effective modal masses of the first 5 modes of vibration of the frames.

Table 4.1: Periods and Effective Modal Masses of Control Frame

	mode				
Control Frame	*1*	*2*	*3*	*4*	*5*
Period (s)	1.1537	0.3687	0.2096	0.1492	-
Effective Modal Mass (%)	89.30	8.36	1.95	0.38	-
Rubberized Concrete Base Isolated Frame	1	2	3	4	5
Period (s)	1.800	0.4709	0.2405	0.1609	0.1304
Effective Modal Mass (%)	97.52	2.02	0.03	0.09	0.12

4.1.1 Storey Drifts

The floor displacement time histories for all frames are shown in Figures 4.2 – 4.26. The time histories indicate that the control frame was well designed and performs well for all reference earthquakes scaling (0.28g to 0.44g peak ground acceleration). A summary of interstorey drifts and overall roof drifts is also presented in Table 4.2.
EN1998-1:2004 set limits to damage limitation for three classes of buildings. For buildings without non-structural elements or with non-structural elements attached to the structure in ways that prevent interference with the structural deformations;

$$d_r v \le h/100;$$

Where; $d_r$is design interstorey drift, *h* is storey height and *v* is reduction factor which takes into account the lower return period of the "service level earthquake". A recommended value of 0.5 is given for buildings of important classes I and II. The National Earthquake Hazards Reduction Programme (NEHRP) also defines an allowable maximum roof drift for its own defined *Group II* building as;

$$\text{Maximum roof drift} = 1.5\%H$$

Where; H is the overall height of the building (NEHRP FEMA 450:Part 1, 2003).

Referring to Table 4.2, the total height of the control frame is 14.2m while that of the base-isolated model is 15.7m. This implies an NEHRP recommendation of 21.3m maximum roof drift for the control frame and 23.6m for the base-isolated model. These values are greater than the roof drifts specification. To minimise this drift, the stiffness can be mobilised by either increasing the column cross-sections or applying braces to the frame, whichever is most economical.

***Table 4.2:** Summary of Interstorey drifts for all Frame Models*

Structure	**Input Node on Drain-2dX**	**Peak Ground Acceleration (m/s^2)**					**Remarks on Damage Limitation Requirement $d_r v \le h/100$**
		0.28g	0.30g	0.34g	0.40g	0.44g	
Control Frame		*Interstorey drifts (cm)*					
Ground	1010	-	-	-	-	-	-
2nd Floor	2010	5.10	5.33	5.81	6.02	8.00	Ok
3rd Floor	3010	4.40	4.59	5.17	5.95	6.79	Ok
4th Floor	4010	3.05	3.22	3.52	3.70	4.23	Ok
5th Floor (roof)	5010	1.72	1.82	1.94	2.32	2.82	Ok
Overall roof drift	**5010**	**13.6**	**14.4**	**16.0**	**17.7**	**19.3**	Meets NEHRP requirement
Rubberized Concrete Base-Isolated Frame							
Ground Floor	2020	10.2	11.1	12.5	15.00	16.60	Soft storeyDeformableFoundation)
2nd Floor	3020	1.67	1.76	1.94	2.20	2.37	Ok
3rd Floor	4020	1.23	1.30	1.44	1.63	1.75	Ok
4th Floor	5020	0.93	1.00	1.09	1.23	1.33	Ok
5th Floor	6020	0.55	0.56	0.66	0.79	0.86	Ok
Overall roof drift	**6020**	**14.5**	**15.5**	**17.6**	**20.7**	**22.7**	Meets NEHRP requirement

***Note**: The National Earthquake Hazard Reduction Programme (NEHRP) limit for maximum roof drift for Group II is 0.015H: where H is the total height of the building.*

EN1998-1:2004 provides that second-order (P-delta) effects can only be neglected if the following requirement is met for all storeys:

$$\Theta = \frac{P_{tot}.d_r}{V_{tot}.h} \leq 0.10 \ \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.\ (3.6)$$

Where: θ is interstorey drift sensitivity coefficient, $P_{tot}$is total gravity load at and above the storey in consideration, $d_r$is the interstorey drift, $V_{tot}$is the storey shear force from the seismic demand, and h is the height of the storey. For values of $0.1 < \theta < 0.2$, EN1998-1:2004 provides that the seismic action effect be amplified by 1/(1- θ) and design be effected for the amplified action effect.

There is an occurrence of"soft storey" at the foundation for the base-isolated model due to the relative excessive drift at the FRP-confined rubberized concrete foundation columns but this is in fact what is intended. The high deformability of these rubberized concrete elements provides a mechanism that deflects the seismic forces and hence reduces the demand on the superstructure which is analogous to a base isolating system.
Fig. 4.2-4.11 shows time histories for floor displacements of the analytical models for the 5 differently scaled peak ground accelerations:

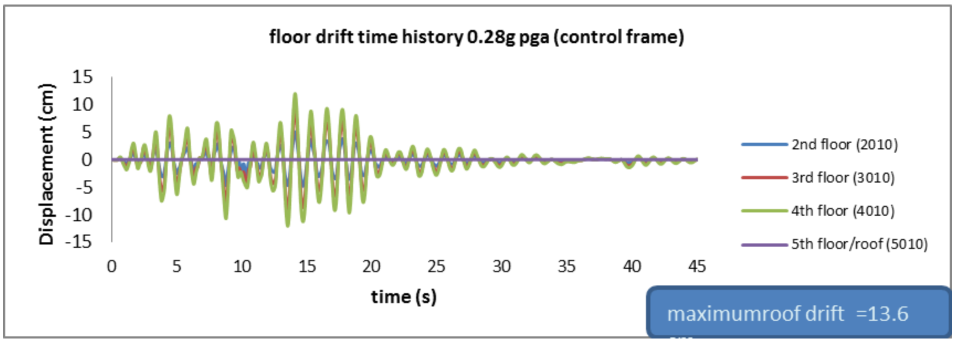

Fig 4.2: Time History of Floor Drift at 0.28 PGA (Control Frame)

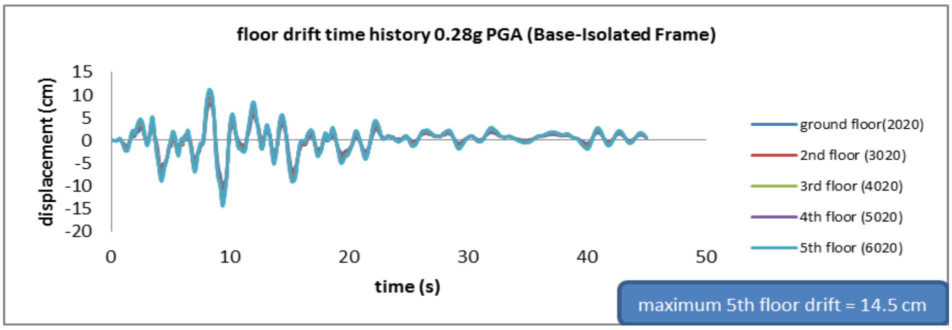

Fig 4.3: Time History of Floor Drift at 0.28 PGA (Base-isolated Frame)

A close study of Fig 4.2-4.3 reveals an interesting finding. The vibration response of the base-isolated frame revealed a more damped system than the control frame. On average, the displacement amplitudes for these analytical models begin to die out rapidly after 10 seconds. The control frame on the other hand experiences maximum displacement amplitudes at 15 seconds and vibration begins to die out only after 20 seconds (Fig 4.2). It is, therefore, reasonable to deduce that the highly damped characteristic of rubberised concrete has reduced the intensity of response for the base-isolated frame. The time-history graph of the base-isolated frame indicates that all floors are moving almost in synchrony. This is a feature of base-isolated structures where the superstructure vibrates as an independent mass with a laterally less-stiff base. The implication of this is an increased period for the fundamental mode, shifting of vibration response to the rights side of the response spectrum and consequent reduction of force demands on the superstructure. The base-isolated frame experiences a little residual drift, say less than 1 cm at the end of 45 seconds as shown in Fig 4.3, Fig 4.5, Fig 4.7 and Fig 4.9.The trend observed in the above paragraph also seems to be the case for the displacement response of the frame models for a peak ground acceleration of 0.30g m/s^2. The general observation is the early attenuation of displacement amplitudes for the base-isolated frame. Figures 4.4- 4.11 show the drift time histories for the different peak ground accelerations:

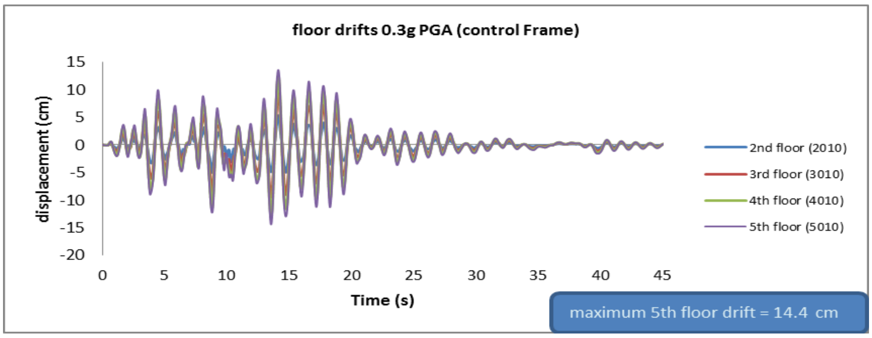

Fig 4.4: Time History of Floor Drift at 0.30 PGA (CONTROL)

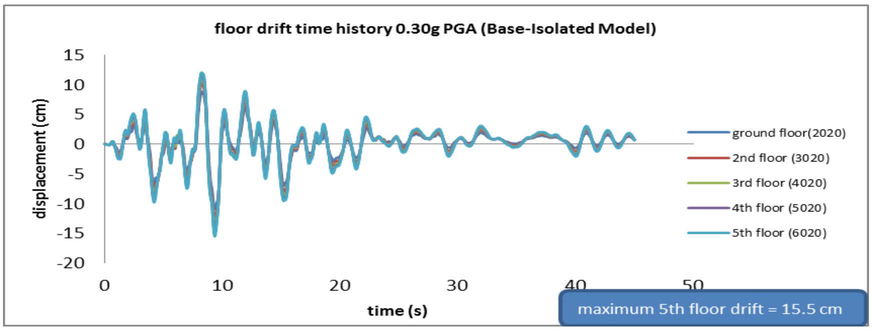

Fig 4.5: Time History of Floor Drift at 0.30 PGA (base-isolated model)

The same trend of response is observed with the rest of the floor displacement time histories. The control (conventional reinforced concrete) frame shows less displacement than the rubberized concrete Base-isolated frame but the latter displayed a more ductile response which is a good material behaviour for seismic design. It indeed responds like a base-isolated structure as desired.

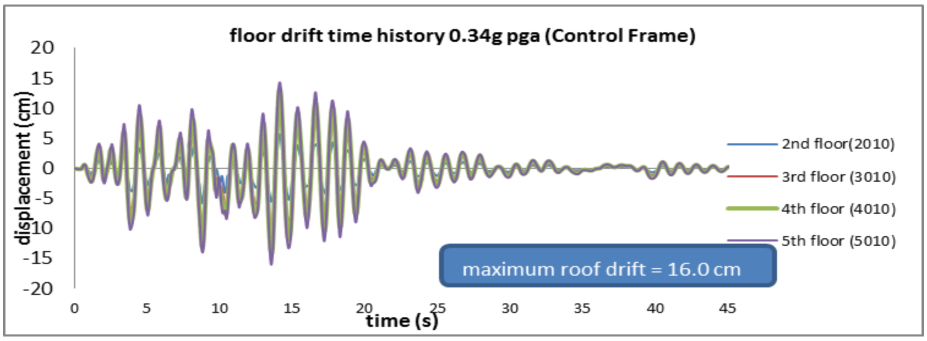

Fig 4.6: Time History of Floor Drift at 0.34 PGA (Control Frame)

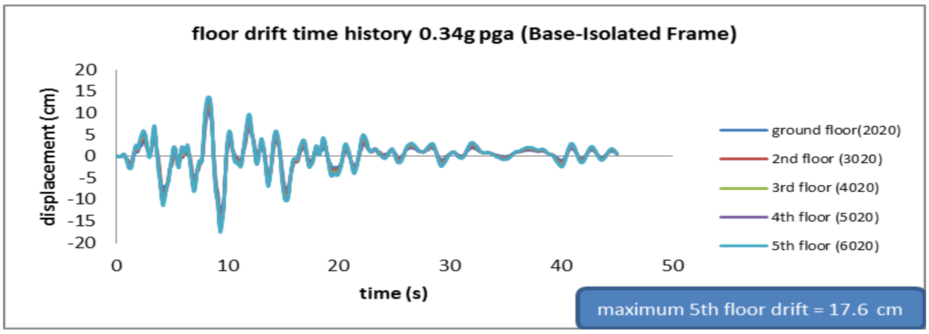

Fig 4.7: Time History of Floor Drift at 0.34 PGA (Base-Isolated Frame)

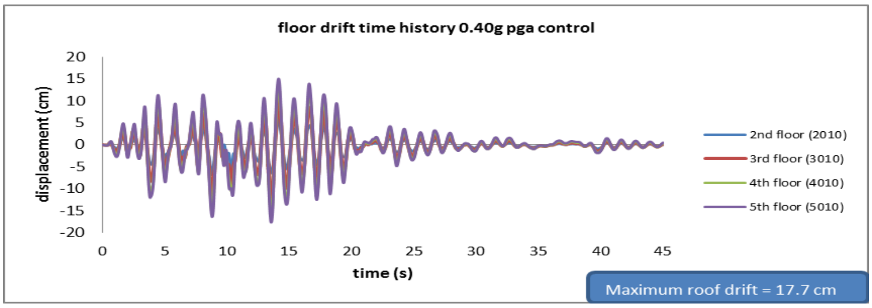

Fig 4.8: Time History of Floor Drift at 0.40 PGA (Control Frame)

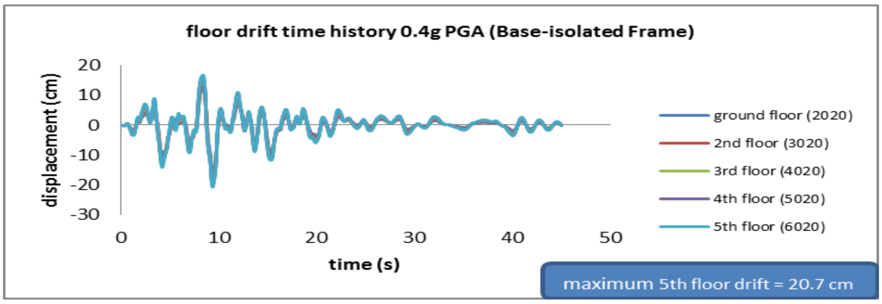

Fig 4.9: Time History of Floor Drift at 0.4 PGA (Base-Isolated Frame)

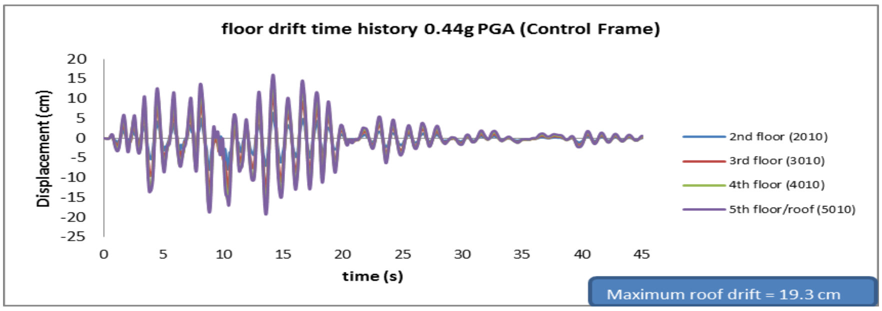

Fig 4.10: Time History of Floor Drift at 0.44 PGA (Control Frame)

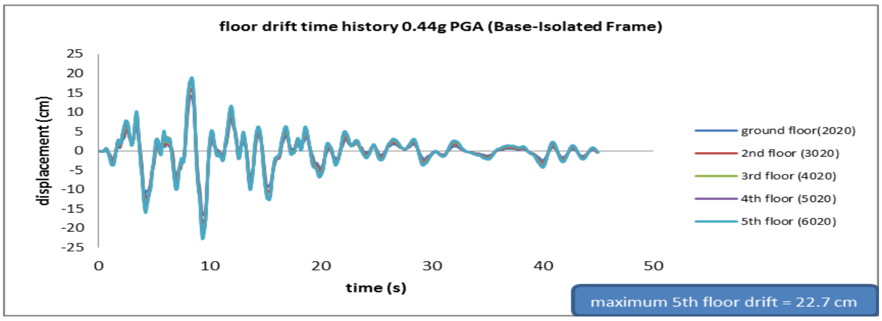

Figure 4.11: Time History of Floor Drift at 0.44 PGA (Base-Isolated Frame)

4.1.2 Structure Ductility

The requirements for ductility in EN1998-1:2004 specifies a behaviour factor of 3.9 for a multi-storey multi-bay structure such as the frame models used. This resulted in a base shear of 1530 kN for the control frame. However, the Rubberized concrete base-isolated model had improved ductility in comparison with the control frame. An attempt can be made to quantify the behaviour factor (*q*-factor) of the models by using equation (3.5) and the *design response spectrum* (Figure 3.12).

From equation 3.5,

$$S_g(T) = a_g.S.\frac{2.5}{q} \, x \, \frac{T_C}{T^2} \, x \, T_D \ldots\ldots\ldots\ldots\ldots \; (3.5)$$

And the parameters of which are determined from the design spectrum of Fig 3.3 are presented as follows:

$S_g(T) = 0.118g$: $a_g = 0.44g$: $S = 1.2$(for ground type B): $T_C = 0.48s$: $T_D = 1.92$ and $T = 1.800s$(for model D).

Solving the above equation (3.5) for q gives a value of 3.2.

This implies that the rubberized deformable foundation provides a *q*-factor of about 3 to the base-isolated structure. The high ductile response reduced the base shear demand of the structure from 1530 kN to 510 kN (69% average reduction in base shear).

Combining the *q*-factor provided by EC-8 requirements (*q = 3.9)* for ductile design and the added *q*-factor derived from the enhanced ductility (*q = 3.2*) gives a total *q*-factor of about 7.

On how accurate the hysteretic behaviour of the deformable foundation elements has been predicted by the Drain-2DX element software used for the analysis, an experimental loading test on large numbers of rubber isolators at the Earthquake Engineering Research Center (EERC) California as reported by [32] shows a good semblance in cyclic response for the Base-Isolated Frame. A comparison between Fig 4.12 and Fig 4.13 shows that the Drain-2DX software analyses are fair.

The global structure hysteretic curves are not presented in this study due to lack of space.

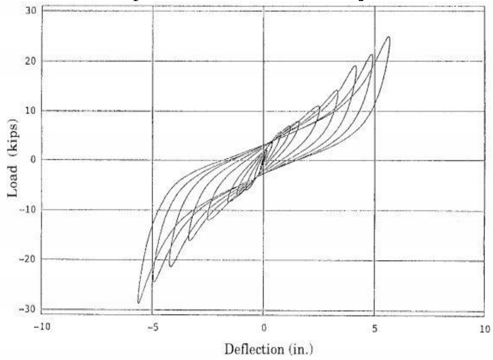

Fig 4.12: *Cyclic loading of High-damping Natural Rubber Isolators (Hysteretic Behaviour) [32]*

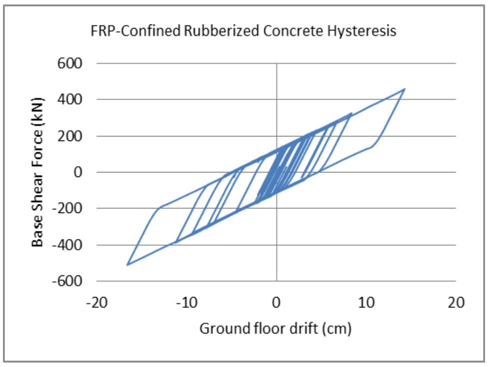

Fig 4.13: *Hysteretic behaviour of 21 FRP-Confined deformable foundationcolumn elements working together at the foundation under the simulated seismic excitation at 0.4g Peak Ground Acceleration.*

4.1.3 Base Shear

A summary of the base shear for both frames is displayed in Table 4.3. Comparison of base shear force for the models shows an increase in base shear with an increase in the seismic force. Interestingly, the base shears for the model on the rubberized deformable foundation system is significantly low as expected. There is a 70% reduction in the base shear demand on the structure. The factors responsible for this desired structural response include:

- Increased redundancy of the frames due to extra load paths provided by the rubberised deformable foundation system;
- Increased ductility due to the highly deformable property of the rubberised concrete leading to an improved non-linear ductile response;
- Increased energy dissipation through rotation and flexure which concentrated damage on the rubberized foundation system and
- Partial Isolation of the superstructure from the direct impact of the seismic.

Table 4.3: Comparison of Base Shear for the frame models

Peak ground acceleration of Simulated Earthquake Time History		CONTROL FRAME	FRAME on FRP-Confined Rubberized Concrete Foundation
0.28g	Base shear Force (kN)	1210	360
	Maximum Roof drift (cm)	13.6	14.5
	Percentage reduction of Base shear (%)	-	70.2
0.30g	Base shear Force (kN)	1240	380
	Maximum Roof drift (cm)	14.4	15.5
	Percentage reduction of Base shear (%)	-	69.4
0.34g	Base shear Force (kN)	1350	420
	Maximum Roof drift (cm)	16.0	17.6
	Percentage reduction of Base shear (%)	-	69.0
0.40g	Base shear Force (kN)	1450	470
	Maximum Roof drift (cm)	17.7	20.7
	Percentage reduction of Base shear (%)	-	67.5
0.44g	Base shear Force (kN)	1530	510
	Maximum Roof drift (cm)	19.3	22.7
	Percentage reduction of Base shear (%)	-	66.7

4.1.4 Energy Dissipation

Figs 4.14-4.15show the energy dissipation by the element groups in the frame models for a peak ground acceleration of 0.40g. A comparison between the base-isolated frame and the control frame reveals an interesting finding. More of the energy was dissipated through the columns which may not be good structural behaviour, however, it is worthy of note that the source of energy dissipation for the base-isolated frame is the FRP-confined rubberized foundation columns and not the columns of the frame. This can be verified from the damage analysis in section 4.1.5. The superstructure of the base-isolated model displayed the least energy dissipation. This is because the FRP-confined rubberized foundation columns responded like a base isolation system, hence reducing the seismic force demand on the superstructure.

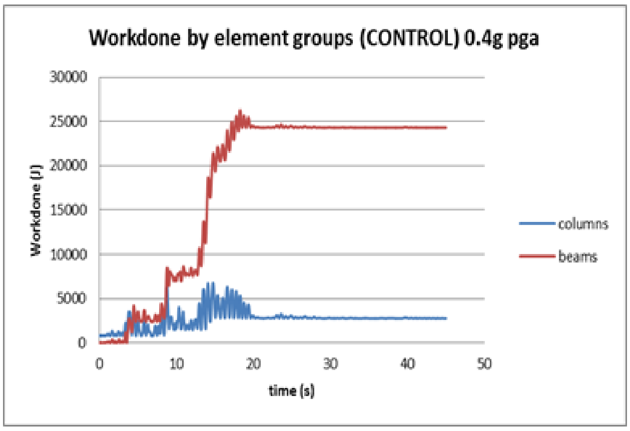

***Fig 4.14**: Energy dissipationtime history of Control Frame*

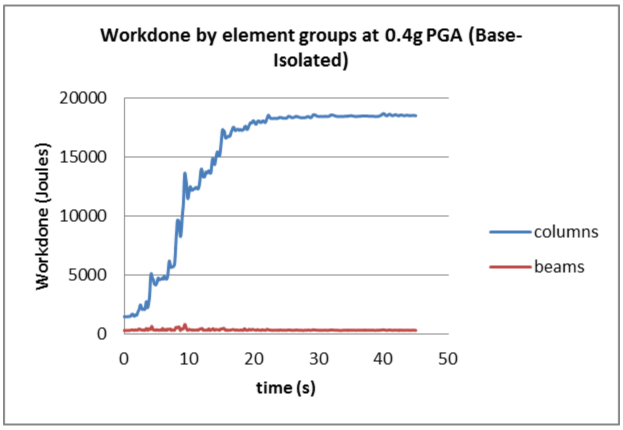

***Fig 4.16:** Energy dissipation time history of Base-Isolated Frame*

4.1.5 *Damage in Structure*

A study of the formation of plastic hinges show that the Base-Isolated frame showed no damage in its superstructure since there was a reduction in the demand on the frame. Damage was concentrated on the deformable foundation elements which are the desired structural behaviour. Fig 4.17 shows the level of damage for each frame model at peak ground acceleration of 0.4g. The red dots on the frames represent plastic hinges. It can, therefore, be proposed that a non-seismic frame can safely be founded on an FRP-confined rubberized concrete deformable foundation and perform satisfactorily. This will pave the way for an innovative, simple-to-design, more affordable and equally efficient alternative to the conventional earthquake-resistant reinforced concrete frames.

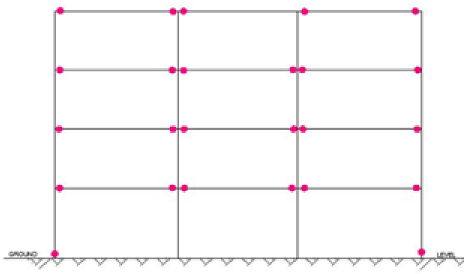

Fig 4.17a: Comparison of Plastic hinges formation in Frames at 0.40g PGA (Control Frame)

Fig 4.17b: Comparison of Plastic hinges formation in Frames at 0.40g PGA (Base-Isolated Frame

V. Conclusion

From the results and findings of this research, the following conclusion is made:

- Using confined rubberized concrete as short foundation columns can lower shear demand on a structure by 70%, and acceleration demand can be reduced by 75 %,
- The reduction in seismic demand in the Base-isolated Frame was mainly due to deflection of the earthquake force by the deformable foundation system in a manner analogous to base isolation systems,
- The FRP-Confined Rubberized concrete foundation performed efficiently to reduce base shear and had the least interstorey drift,
- A less expensive frame designed according to the provisions of Eurocode-2 can perform satisfactorily if founded on FRP-confined rubberized concrete columns foundation system due to the resulting small acceleration demand,
- The reduction in demand implies less material for the superstructure which implies lesser cost of shelter.
- FRP-confined rubberized concrete can be used to develop quick, simple, and affordable houses for the impoverished people in developing (and earthquake-prone) regions of the world,
- The proposed deformable system from FRP-confined rubberised concrete is not only cheap, simple to construct with a little technical know-how but is also environmentally friendly. Rubber from waste tyres is not affected by ageing since the constituents are non-biodegradable,
- Finally, incorporating waste vehicle tyre rubber in concrete results in a cleaner and sustainable environment.

5.2 Recommendation

Due to time constraints, only one earthquake time history was used in this research to investigate structural response. Further investigation can be carried out using a good number of recorded earthquake time histories. Future work along this path shall include shake table tests in order to validate the findings of this research.

Acknowledgement

Special thanks to Samar Raffoul of the Department of Civil and Structural Engineering at the University of Sheffield, United Kingdom for providing laboratory data of her research on FRP-Confined rubberized concrete.

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