Non Linear Dynamic Analysis of Vierendeel Girder System Using Time History Analysis

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Abstract- *This paper evaluates the nonlinear procedures used for Performance Based Design, the Nonlinear dynamic procedure (time history).The purpose of this study is to determine in depth understanding of inelastic seismic response of rectangular Rcc buildings with diaphragram with openings. Non-linear time history dynamic analysis is performed using Elcentro data on typical modular structure to assess the seismic performance of these structures. The analysis accounts for their unique detailing especially to elements that contribute to energy dissipation during major seismic events. This study revealed significant global seismic capacity as well as a satisfactory performance at design intensity levels. This paper deals with an investigation carried out by the authors to study the behavior of simple parallel chorded reinforced concrete Vierendeel girders in their elastic as well as ultimate load conditions. Details of tests carried out on ANSYS along with measurements and observations made are fully described. The application of plastic analysis to reinforced concrete Vierendeel girders has been examined and shown to be feasible when the constituent members of the girders possess the desirable flexural characteristics. A simple method is suggested for predicting the exact mode of failure and to calculate the ultimate load. The predominating influence of depth span ratio of the girders over their behavior in service conditions has been demonstrated and the importance of selecting a proper depth span ratio so as to secure the desirable cracking characteristics and effective stiffness has been pointed out. The comparative behavior of Vierendeel girders designed on elastic theory and girders with arbitrary uniform reinforcements has also been investigated and their relative advantages discussed.*

Keywords- Response spectrum, IS 1893:2002 and 2016, Time History Analysis,Vineral Girder, ANSYS

I. INTRODUCTION

1.1 General:

The Belgian Engineer Arthur Vierendeel introduces the Virendeel Girder in 1896.Vierendeel frame or truss is a series of rectangular frames which achieves stability by the rigid connection of the vertical web members to the top and bottom chord. Vierendeel transfer shear from the chords by bending moment in the vertical webs .As a result, all members are combined stress member in which axial, shear, and bending stresses exist. The Vierendeel frame will be heavier than an equivalently loaded truss. Even though the diagonals are eliminated, bending in all members results in chord sizes and vertical webs significantly larger in cross-sectional area. Vierendeel appears highly advantageous where mechanical requirements are extensive and required to accommodate large duct work or elbow room to change direction. In a structural system, the Vieredeel gain tremendous rigidity with increase depth several stories of a Vierendeel grid linked together.

Safety in structural design is typically addressed by considering uncertainties in both the structure and the expected loads and introducing safety factors in the design process. A number of engineering failures, however, are related to accidental loading effects that are difficult to quantify and incorporate in the original design. Because the progressive collapse resulting from the gas explosion in the Ronan Point Apartment Building in England in 1968,1 widespread attention has been given to collapse-resistant performance of structures.2 The threat of terrorism has further highlighted the need to explicitly consider collapse resistance particularly for critical and public structures. Several existing British and U.S. codes provide limited guidelines for considering progressive collapse resistance of structures in the design process.

1.2 Motivation and Objectives

Although numerous publications have dealt with the behavior and design of concrete diaphragms, it is clear that there are several issues that have not yet been resolved.

Openings in diaphragms are often unavoidable and their presence can significantly modify the behavior of the diaphragm. At present and in many cases the designer assumes that the diaphragm is a rigid element, totally ignoring in-plane deformations – an assumption that can lead to erroneous results. Nor is it satisfactory to assume that the diaphragm acts as a continuous elastic beam over the shear walls and frames running in the transverse direction for low-rise rectangular buildings with longer floor aspect ratios (greater than 3:1 ratio) without accounting for in-plane nonlinear deformation of the diaphragms. It is possible that the lateral load distribution of diaphragm inertia forces to the vertical frame elements may be compromised in a manner yielding an outcome contrary to what is assumed. This issue is considered vitally important, as it is the least understood subject in this area, since there is no quantification of the error in diaphragm and frame shears as a result of ignoring openings. Therefore, a systematic study of a set of carefully devised scenarios covering a spectrum of typical configurations is crucial where diaphragm in-plane deformations are incorporated in the analysis in order to capture the "real" behavior of the structural members as opposed to the "assumed" one.

Even though a total collapse of the diaphragm is unlikely to be the first major event in the failure of a building, a deterioration of its stiffness may result in a shift in the lateral loads distribution to the load carrying vertical elements causing some members to be overloaded resulting in a failure at that locality, thus jeopardizing the safety of the building structure and compromising the expected diaphragm action.

The proposed research will investigate the aforementioned issues in depth and will offer pertinent insights and better understanding of the structural behavior and design of RC buildings with floor diaphragm openings when subjected to strong ground motion. The main goal of this research effort is to gain in-depth understanding of inelastic seismic response of rectangular RC buildings with diaphragms with openings through the following

1.5 Aim and Objectives

Aim

This study aims for non linear dynamic analysis of vierendeel girder system using time history analysis.

Objective:

1. To enhance IDARC2 [56] -developed in 1988- to account for RC buildings with diaphragm openings. Special attention will be given to the algorithms used in obtaining the in-plane idealized moment-curvature curves from the current fiber model.
2. To investigate the influence of estimated hysteretic parameters for slabs with openings.
3. To investigate the applicability of rigid floor assumption (neglecting their in-plane deformations) to modeling of floor diaphragms with openings of various sizes placed in symmetric and asymmetric plan locations. Also, to investigate the influence of floor diaphragms on the distribution of lateral loads among the frames and shear walls considering the floors' inelastic-in-plane deformations. This will result in establishing a criterion as to when floor diaphragm openings in earthquake resistance design of RC rectangular buildings with shear walls can be ignored.

Hence, by using a suite of actual earthquake accelerations as ground motion input for the dynamic analysis, the true behavior of the diaphragm will be better captured, which will lead to a deeper understanding of diaphragm behavior during a seismic event in RC buildings with flexible (elastic and inelastic) diaphragms with openings. It will also provide a timely and enhanced computational tool for the research community to use.

II. LITERATURE REVIEW

LITERATURE REVIEW

Lei Zhang1 ,Hailong Zhao1,2,* , Tiecheng Wang1,2 and Qingwei Chen1 'Parametric Analysis on Collapse-resistance Performance of Reinforced-concrete Frame with Specially Shaped Columns Under Loss of a Corner Column' 2016

A finite element model is verified accurate enough to simulate the static test of one reinforced-concrete frame with specially shaped columns subjected to the loss of a ground corner column. As the frame sustained loads primarily depending on the beam resisting mechanism in the test, four related parameters, namely the height of beam section, rebar ratio of beam, rebar ratio of slab and limb length of specially shaped column are chosen for parametric analyses respectively. It is indicated that the collapse resistance capacity remarkably increases with the increasing of the height of beam section and rebar ratio of the lower steel bars of beam. The increase of the rebar ratio of slab and upper steel bars of beam could enhance the stiffness and collapse-resistance capacity slightly. The lengthening of the limb of specially shaped column only increases the stiffness of the frame. According to an equivalent method, the rectangular column frame is obtained to compare collapse-resistance performance with the specially shaped column frame. It is concluded that the frame with specially shaped columns could maintain the equivalent collapse-resistance capacity while reduce the lateral stiffness compared with the rectangular column frame

Jun Yu and Kang Hai Tan 'Experimental study on catenary action of RC beam-column sub-assemblages' 2010:

Catenary action is considered as the last defense of a structure to mitigate progressive collapse, provided that the remaining structure after an initial damage can develop alternate load paths and a large deformation has occurred in the affected beams and slabs. As a result, catenary action requires high continuity and ductility of joints. To investigate whether current RC structures designed according to ACI 318-05 could develop catenary action under column removal scenarios, two one-half scaled beam-column sub-assemblages with seismic and non-seismic detailing were designed and tested to complete failure, i.e. rebar fracture. The sub-assemblage consists of two end column stubs, a two-bay beam, and one middle beam-column joint at the junction of two single-bay beams. To ensure sufficient horizontal resistance, the sizes of end columns were enlarged to be rather stiff. To simplify the boundary conditions in the first batch of tests of our ongoing project and to make the test system statically determinate, two end column stubs were supported onto two horizontal restraints and one vertical restraint to simulate the encased supports. A concentrated load was applied vertically by a hydraulic actuator on the top of the middle joint using displacement control until the whole system eventually failed. The loading rate was controlled manually to simulate quasi-static structural behavior. The study provided insight not only into catenary action of sub-assemblages, but also the performance and failure mode of the middle joints, as well as the influence of two different detailing requirements. During the whole loading history, the cross-sectional internal forces at any beam locations can be evaluated according to the measured reaction forces. Finally, a simple analytical model will be used to check the mechanism of catenary action.

E. Wahyuni, Y. Tethool'

Effect of vierendeel panel width and vertical truss spacing ratio in staggered truss framing system under earthquake loads' June 2015:The purpose of this study is to determine the effect of vierendeel panel width and vertical truss spacing ratio in an inelastic behavior of the STF system due to earthquake loads. The STF system is applied to a six-storey building that serves as apartments. The STF system is used in the building in the transverse direction (N-S direction), while in the longitudinal direction (W-E direction) the building system uses the special moment resisting frame. The structural behavior was evaluated using nonlinear pushover and time history analyses. The results showed that by increasing the ratio of vierendeel panel width and vertical truss spacing, the ductility of the structure was increased. Based on the performance evaluation, the ratio of the vierendeel panel width and vertical truss spacing on the STF buildings that provided satisfactory performance was more or equal to 1.6. The ultimate drift obtained from non-linear time history analysis was smaller than the pushover analysis. This result showed that the static nonlinear pushover analysis was quite conservative in predicting the behavior of the six-storey building in an inelastic condition.

III. METHODOLOGY

3.1 The finite part analysis

May be a numerical technique. During this technique all the complexities of the issues, like variable form, boundary conditions and masses are maintained as they are they're however the solutions obtained are approximate. Attributable to its diversity and adaptability as Associate in Nursing analysis tool, it's receiving abundant attention in engineering. The quick enhancements in component technology and dynamic of price of computers have boosted this technique, since the pc is that the basic want for the applying of this technique. Variety of in style whole of finite part analysis packages is currently accessible commercially. a number of the popular packages are STAAD-PRO, GT-STRUDEL, NASTRAN, NISA and ANSYS. Victimization these packages one will analyze many complicated structures. The finite element analysis originated as a method of stress analysis in the design of aircrafts. It started as an extension of matrix method of structural analysis.

Civil engineers use this method extensively for the analysis of beams, space frames, plates, shells, folded plates, foundations, rock mechanics problems and seepage analysis of fluid through porous media. Both static and dynamic problems can be handled by finite element analysis.

3.1. Description of Method

In engineering issues there square measure some basic unknowns. If they're found, the behaviour of the complete structure may be expected during a time, these unknowns square measure infinite. The finite component procedure reduces such unknowns to a finite range by dividing the answer region into little components known as parts and by expressing the unknown field variables in terms of assumed approximating functions (Interpolating functions/Shape functions) inside every component. The approximating functions square measure outlined in terms of field variables of specified points known as nodes or nodal points. So, within the finite component analysis the unknowns square measure the sphere variables of the nodal points. once choosing parts and nodal unknowns next step in finite component analysis is to assemble component properties for every component. Maybe, in solid mechanics, we've got to seek out the force-displacement i.e. stiffness characteristics of every individual component. Mathematically this relationship is of the shape.

3.2. Finite Element Analysis

Where [k]e is element stiffness matrix, {δ} eis nodal displacement vector of the element and {F}"e" is nodal force vector. The element of stiffness matrix misrepresent the force in coordinate direction 'i' due to a unit displacement in coordinate direction 'j'. Four methods are available for formulating these element properties viz. direct approach, Element properties are used to assemble global properties/structure properties to get system equations [k] {δ} = {F}. Then the boundary conditions square measure obligatory. The answer of that equation offers the nodal unknowns. Using these nodal values extra calculations square measure created to induce the specified values e.g. stresses, strains, moments, etc. in solid mechanics issues. Thus, the various steps involved in the finite element analysis are:

(i) Select suitable field variables and the elements.
(ii) Discredited the continua.
(iii) Select interpolation functions.
(iv) Find the element properties.
(v) Assemble element properties to get global properties.
(vi) Impose the boundary conditions.
(vii) Solve the system equations to get the nodal unknowns.
(viii) Make the additional calculations to get the required values

3.3. Structural Design Consideration

3.3.1. Loading and Failure Mechanisms

A shear wall is stiffer in its optic axis than it is within the alternative axis. it's thought of as a primary structure that provides comparatively stiff resistance to vertical and horizontal forces acting in its plane. Beneath this combined loading condition, a shear wall develops compatible axial, shear, tensional and flexural strains, leading to an advanced internal stress distribution. During this approach, masses area unit transferred vertically to the building's foundation. Therefore, there are unit four vital failure mechanisms; as shown in Figure one. The factors decisive the failure mechanism embody pure mathematics, loading, material properties, restraint, and construction.

3.3.2. Slenderness Ratio

The 'slenderness ratio' of a wall is outlined as a perform of the effective height divided by either the effective thickness or the radius of the gyration of the wall section. It's extremely regarding the 'slenderness limit' that's the cut-off between components being classed 'slender' or 'stocky. The slender walls area unit susceptible to buckling failure modes, as well as Leonhard Euler in-plane buckling thanks to axial compression, Leonhard Euler out-of-plane buckling thanks to axial compression and lateral torsional buckling thanks to bending moment. Within the style method, structural engineers ought to think about of these failure modes to make sure that the wall style is safe underneath varied sorts of doable loading conditions.

3.3.3. Coupling effect of shear walls

In actual structural systems, the shear walls might operate as a coupled system rather than isolated walls counting on their arrangements and connections. 2 neighbor wall panels will be thought of coupled once the interface transfers longitudinal shear to resist the deformation mode. This stress arises whenever an area experiences a flexural or restrained deformation stress and its magnitude relies on the stiffness of the coupling component.

Counting on this stiffness, the performance of a coupled section can fall between that of a perfect uniform component of comparable gross arrange cross-sectional and also the combined performance of the freelance part components. Another advantage of coupling is that it enhances the general flexural stiffness dis-proportionally to shear stiffness, leading to smaller shear deformation.

3.4. Methods of Seismic Analysis of Building

General: Earthquakes area unit nature's greatest hazards to life on this planet. The hazards obligatory by earthquakes area unit distinctive in several respects, and consequently going to mitigate earthquake hazards needs a novel engineering approach. a crucial distinction of the earthquake drawback is that the hazard to life is associated virtually entirely with manmade structure expect for earthquake triggered landslides, the sole earthquake impact that causes in depth loss of life area unit collapse of bridges, buildings, dams, and alternative works of man. This facet of

earthquake hazard may be countered solely by styles and construction of earthquake resistant structure. The optimum engineering approach is to style the structure therefore on avoid collapse in most doable earthquake, so guaranteeing against loss of life however accretive the chance of harm.
Various methods for determining seismic forces in structures fall into two distinct categories:

(i) Equivalent static force analysis
(ii) Dynamic Analysis

IV. MODELLING AND ANALYSIS ANSYS

The definition of the proposed numerical model was made by using finite elements available in the ANSYS code default library. SOLID186 is a higher order 3-D 20-node solid element that exhibits quadratic displacement behavior. The element is defined by 20 nodes having three degrees of freedom per node: translations in the nodal x, y, and z directions. The element supports plasticity, hyper elasticity, creep, stress stiffening, large deflection, and large strain capabilities. It also has mixed formulation capability for simulating deformations of nearly incompressible elastoplastic materials, and fully incompressible hyper elastic materials. The geometrical representation of is show in SOLID186 fig 3.5

This SOLID186 3-D 20-node homogenous/layered structural solid were adopted to discrete the concrete slab, which are also able to simulate cracking behavior of the concrete under tension (in three orthogonal directions) and crushing in compression, to evaluate the material non-linearity and also to enable the inclusion of reinforcement (reinforcement bars scattered in the concrete region).The element SHELL43 is defined by four nodes having six degrees of freedom at each node. The deformation shapes are linear in both in-plane directions. The element allows for plasticity, creep, stress stiffening, large deflections, and large strain capabilities. The representation of the steel section was made by the SHELL 43 elements, which allow for the consideration of non-linearity of the material and show linear deformation on the plane in which it is present. The modelling of the shear connectors was done by the BEAM 189 elements, which allow for the configuration of the cross section, enable consideration of the non-linearity of the material and include bending stresses as shown in fig 3.4. CONTA174 is used to represent contact and sliding between 3-D "target" surfaces (TARGE170) and a deformable surface, defined by this element. The element is applicable to 3-D structural and coupled field contact analyses. The geometrical representation of CONTA174 is show in fig 3.1. Contact pairs couple general axisymmetric elements with standard 3-D elements. A node-to-surface contact element represents contact between two surfaces by specifying one surface as a group of nodes. The geometrical representation of is show in TARGET 170 fig 3.2.

The TARGET 170 and C0NTA 174 elements were used to represent the contact slab-beam interface. These elements are able to simulate the existence of pressure between them when there is contact, and separation between them when there is not. The two material contacts also take into account friction and cohesion between the parties.

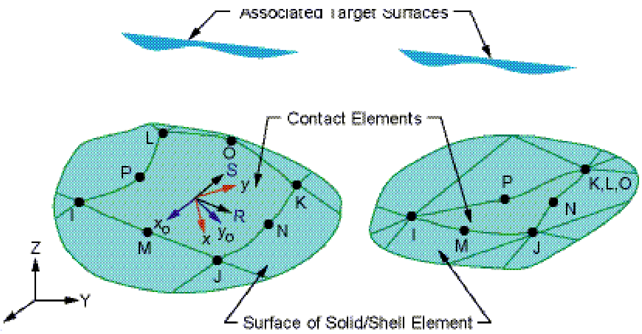


Fig.1: CONTA 174

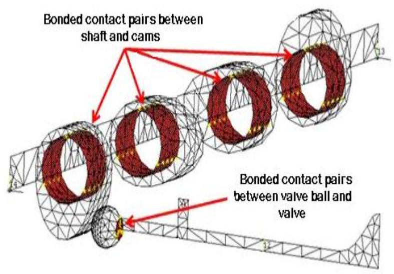


Fig.2: TARGET 170

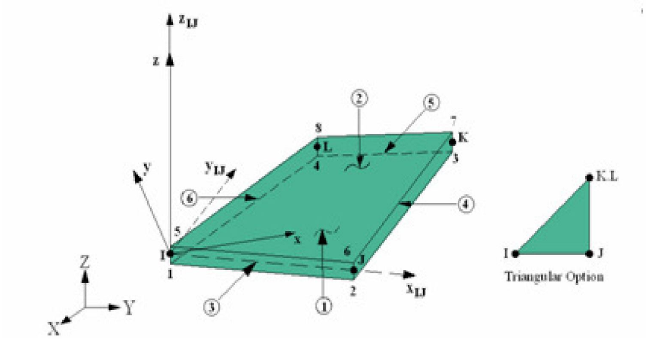


Fig.3: Shell 43

V. PROBLEM STATEMENT

- The span to depth ratio=1/8 to 1/10 are typical.
- Site location:
- Strctural consultant:

- Width of Girder: 4m
- Depth of Girder:4m
- Total span:10m
- Slab Thickness:200mm
- Live load : 3 Mpa
- Dead Load:Self weight=Volume * Density

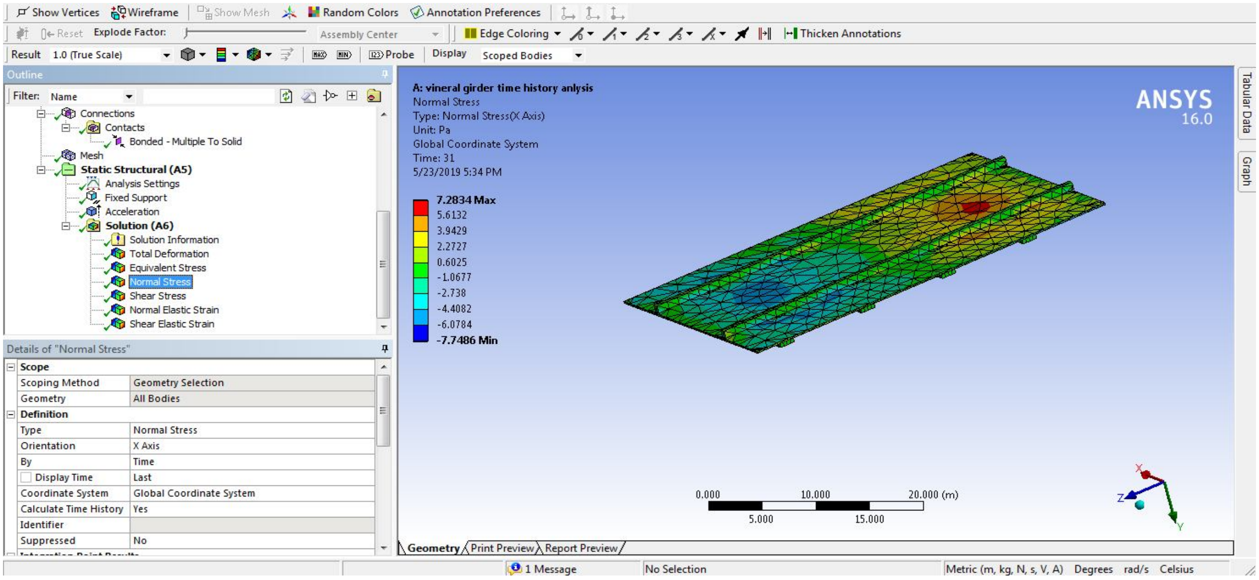


Fig 4 Modeling in ANSYS

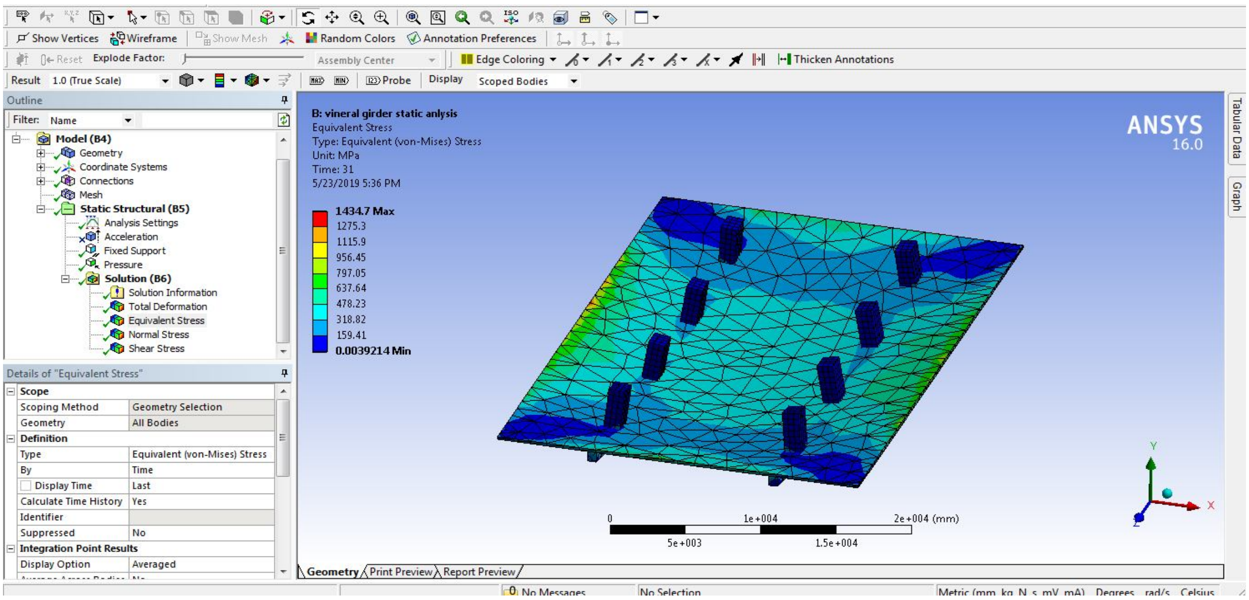


Fig 5 Modeling in ANSYS

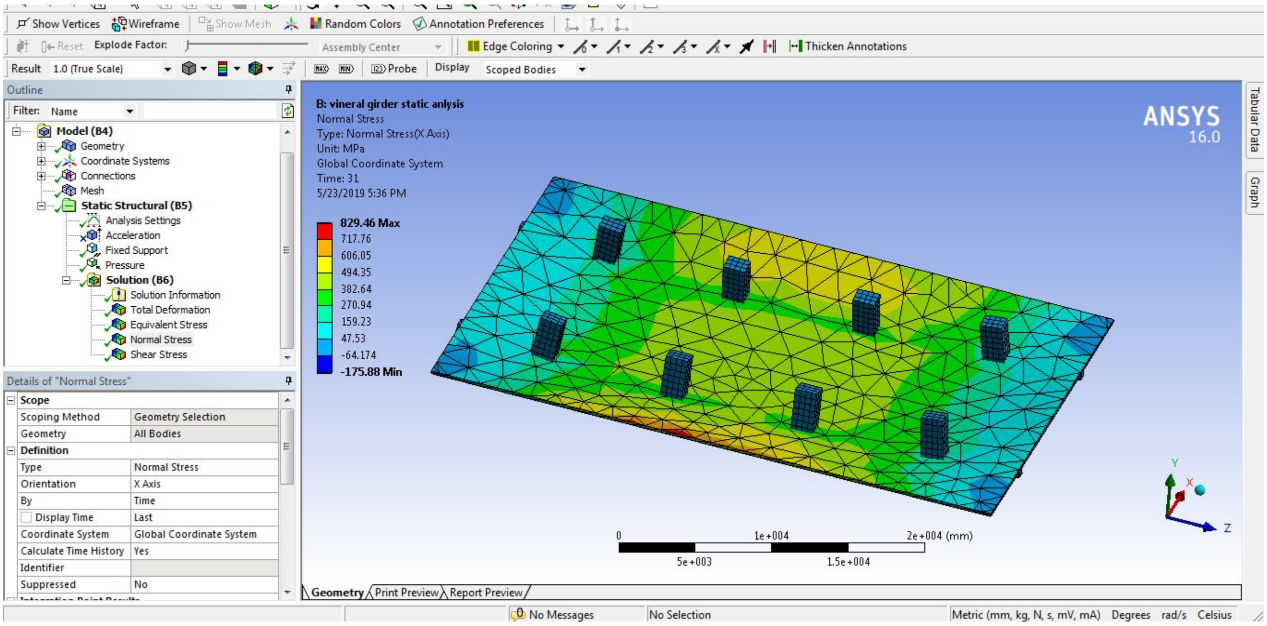
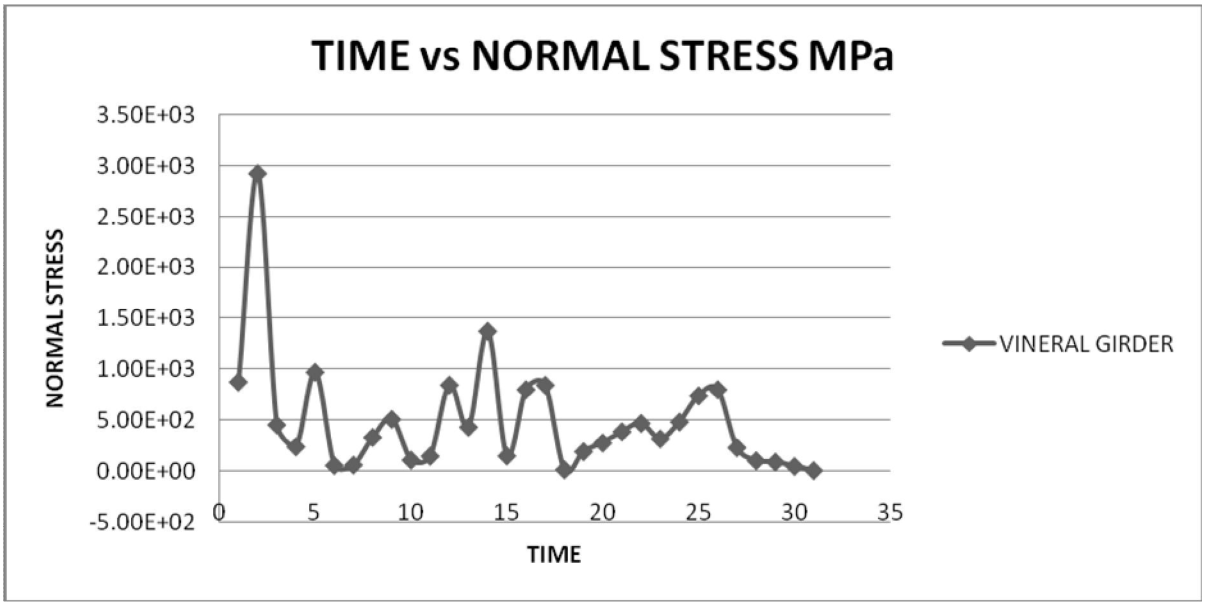


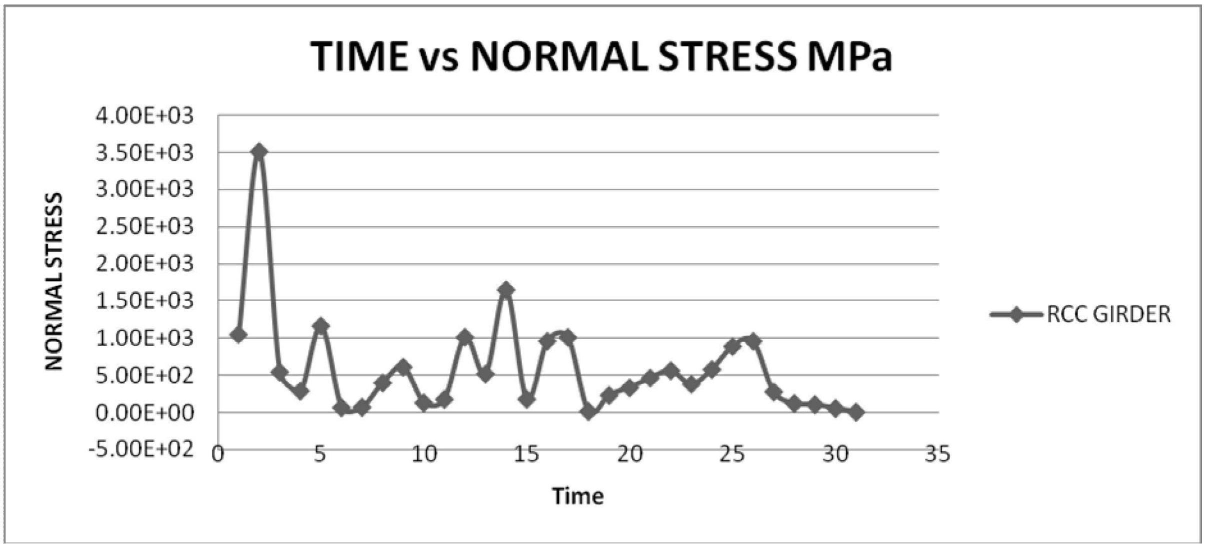
Fig 6 Modeling in ANSYS

VI. RESULT & DISCUSSION

Time Vs. Normal Stress



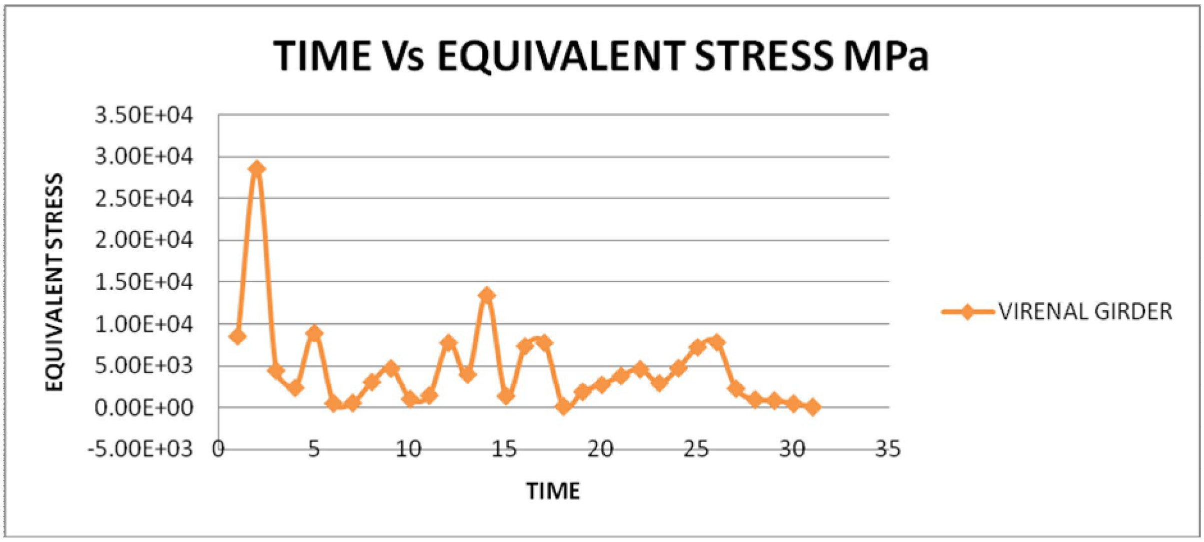
Graph 1 Time Vs. Normal Stress For Vineral Girder



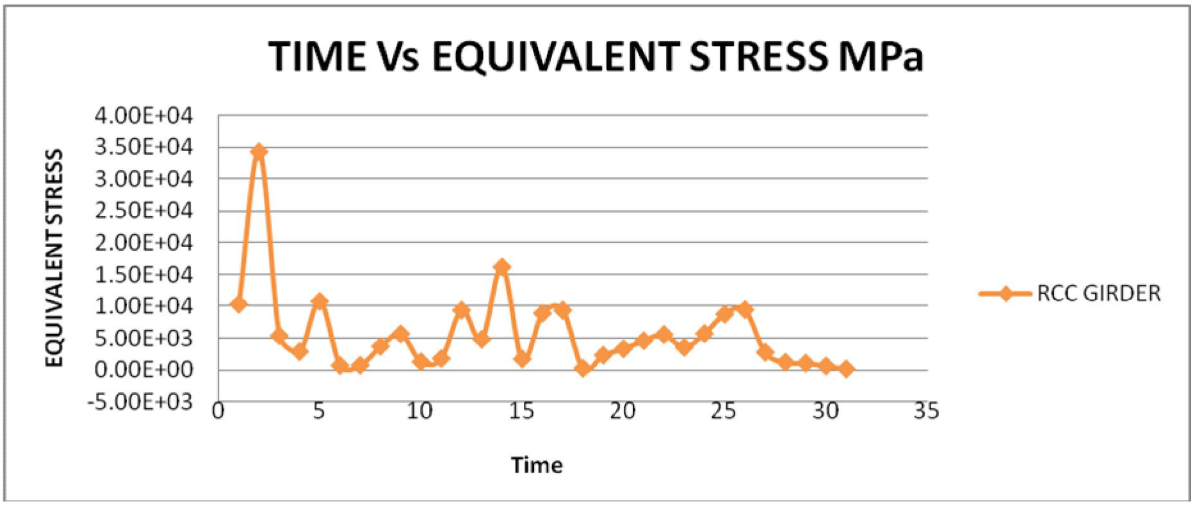
Graph 2 Time Vs. Normal Stress For RCC Girder

From The Above both graphs Shows that Normal stresses in RCC Girder Is 3.50E+03 And in Vineral Girder is 3.00E+03 so it conclude that normal stress in RCC Girder is greater than Vineral girder

Time Vs. Equivalent Stress

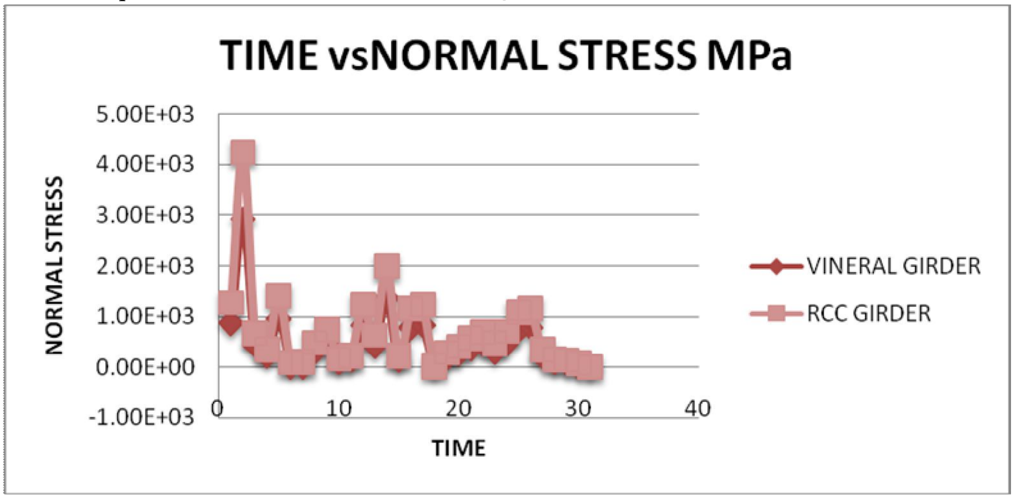


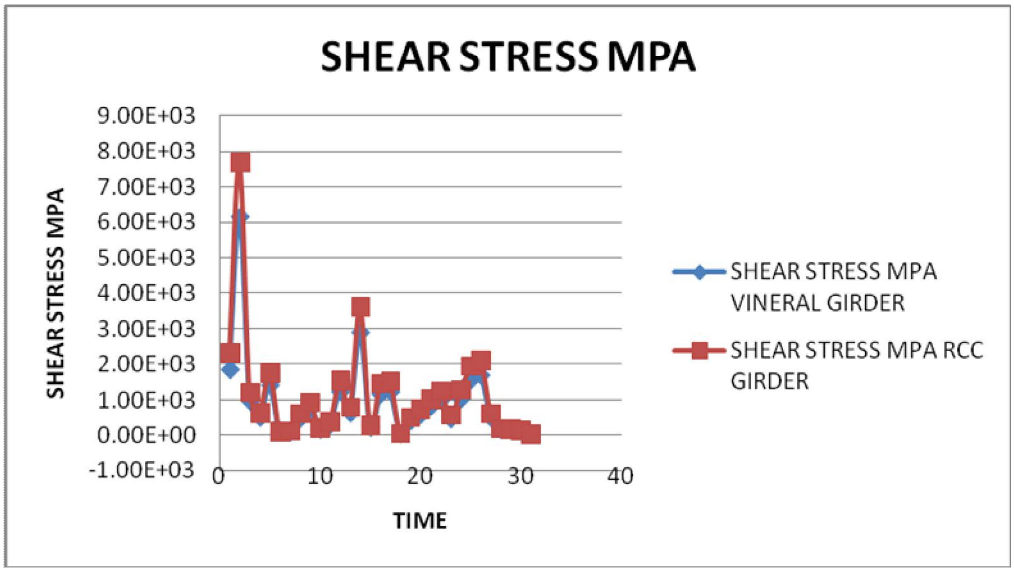
Graph 3Time Vs. Equivalent Stress Fir vineral Girder



Graph 4Time Vs. Equivalent Stress Fir RCC Girder

From The Above both graphs Shows that Equivalent Stress in RCC Girder Is 3.50E+04 And in Vineral Girder is 3.00E+04 so it conclude that Equivalent Stress in RCC Girder is greater than Vineral Girder





VII. CONCLUSIONS

In this paper the RCC vineral girder is analysed and compared with conventional building both static linear and dynamic non linear analysis is done using FEA tool ANSYS. Following conclusions can be made up to now

- The normal stress is 45% less in vineral gireder as compared with conventional beam.
- The total deformation due to elcentro is 34%% less in vineral gireder as compared with conventional beam.
- The shear stress is 31% less in vineral gireder as compared with conventional beam.
- The equivalent stress is observed19% less in vineral gireder as compared with conventional beam.

VIII.FUTURE STUDY

Following parameters are proposed for further work

- The validation of present case study with experimental model
- The analytical comparison using ANSYS for various span to depth ratio as per code provision subjected to Dynamic load

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