TO STUDY THE EFFECT OF SOFT STOREY TO LATERAL STIFFNESS OF REINFORCED CONCRETE STRUCTURES UNDER SEISMIC EXCITATION

Final Year Project UG-2014

By

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Islamabad, Pakistan

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This is to certify that the

Final Year Project Titled

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has been accepted towards the requirements

for the undergraduate degree

in

CIVIL ENGINEERING

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"In the name of Allah, the most beneficent, the most merciful"

We are highly grateful to our supervisor Lecturer Arslan Mushtaq for providing us and enabling us to get deep understanding of earthquake engineering as a core subject. His guidance and continuous encouragement helped us to think beyond the visible facts to get more useful and applicable conclusions from the work in hand. His friendly guidance helped us to discuss our point in detail.

Completion of this research work is the outcome of co-operation of many supportive people. It is hard to measure their assistances. Besides our supervisor we would also like to thank Mr. Muhammad Usman, our co-advisor. Finally we are thankful to our parents who were always supportive and encouraged us during intervals of work stress.

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ABSTRACT

Reinforced Concrete was discovered by Joseph Monier. Due to its speed of construction, flexibility, sustainability and easiness to cast, it became the first choice of civil engineers of 19th century. Many RC structures have been constructed and many others are still in progress. Similarly, in other countries, our country Pakistan has many ancient RC structures. RC structures are subjected to lateral loads due to earthquake and wind. Especially northern areas in Pakistan are subjected to more earthquakes. The structures should be able to resist these lateral loads and suffer least damage during earthquake. As in the high-rise buildings parking is usually provided at the ground floor. Due to more open space requirement for parking purpose the stiffness of the ground floor is comparatively less as compared to upper stories. So this bottom storey behaves as a soft storey.

In order to improve the stiffness of this soft storey for better seismic response various techniques are used like shear walls and CFRPs etc. But a more economical of all these options is Galvanized steel welded wire mesh which is usually wrapped around the reinforcement of beam-column joint region. So, in this way it provides better confinement and hence better seismic performance of RC structures by increasing their ductility.

1 INTRODUCTION

1.1 GENERAL:

Earthquakes have been occurring since a very long time but we understood what earthquakes are and what causes them since last century. Earthquakes result in huge loss of life and economy. Mankind always struggled to minimize the damage caused by earthquakes. Many researchers have explained the nature and hence the resulting damages from earthquakes. All the economic and human losses are mainly because of structures made by human beings like buildings, roads, bridges and dams etc.

In recent past many devastating earthquake events occurred. Some of them occurring throughout the world are Bhuj in India (2001), Bam in Iran (2003),Kashmir in Pakistan with a magnitude of 7.6 (Shahzada et al.2011;Maqsood and Shwarz(2011),China(2010) and Indonesia(2010).

Major losses are because of poor construction practices especially in the developing countries. This is mainly due to non-engineered structures in which design guidelines are not followed or they are mainly designed based on old building codes. Most of these are designed to sustain gravity loads only and are not designed to sustain seismic or earthquake loading. The use of different building codes, planning rules, construction methods and materials mean that their vulnerability changes from one place to another.

Huge loss of lives and economy in the last decade necessitates the strong need for Earthquake Risk Assessment (ERA) for all of the countries having a strong risk. ERA has 3 main components; seismic hazard, vulnerability of exposed structures and the loss (Ahmad 2011) ERA helps in preparedness, planning and policy making in hazards for mitigating harmful effects. Vulnerability is the main component of ERA and damage indicators at different hazard levels are established by it. Different parts of the world differ in vulnerability due to different building codes being followed in that specific area, construction techniques and methods for quality assessments and construction materials.

Therefore earthquake vulnerability needs to be checked for researchers and designers for reducing the damaging effect of earthquake to minimum. South Asian countries including Pakistan is under a radar of moderate to high level earthquake. October 8, 2005 Earthquake in Kashmir has done a catastrophic damage and therefore generating a need for researcher to improve the earthquake behavior of buildings in order to avoid more catastrophic activity like that.

It is completely impossible to fully diminish the effects of earthquake but the damage caused by it can be mitigated by taking suitable measures. Several methods are used for this purpose like shock absorbers and tuned mass dampers. One of the most economical and suitable way is to wrap galvanized steel welded wire mesh inside the reinforcement in the beam-column joint region.

As due to increasing cost of plots and more congestion there is a trend of the high rise buildings. There is parking requirement for such high rise buildings which is usually provided at the ground floor of such high rise buildings. For parking, we need open space so the infill walls or panels are eliminated. By doing so the stiffness of the bottom storey becomes less as compared to upper stories. So during an event of earthquake this storey is more prone to damage and collapse as compared to upper stories. There is a need to improve its stiffness. So for this purpose we have many options available but we have to opt for the most economical one.

Galvanized steel welded wire mesh which is easily available in the market and is also economical is wrapped in the inner side of reinforcement in the beam-column joint region. So in this way both the confinement and ductility is improved resulting in the better seismic performance of the structure.

So keeping in view the whole discussion further research on seismic analysis of galvanized steel welded wire mesh used in beam-column joint region structure is carried out.

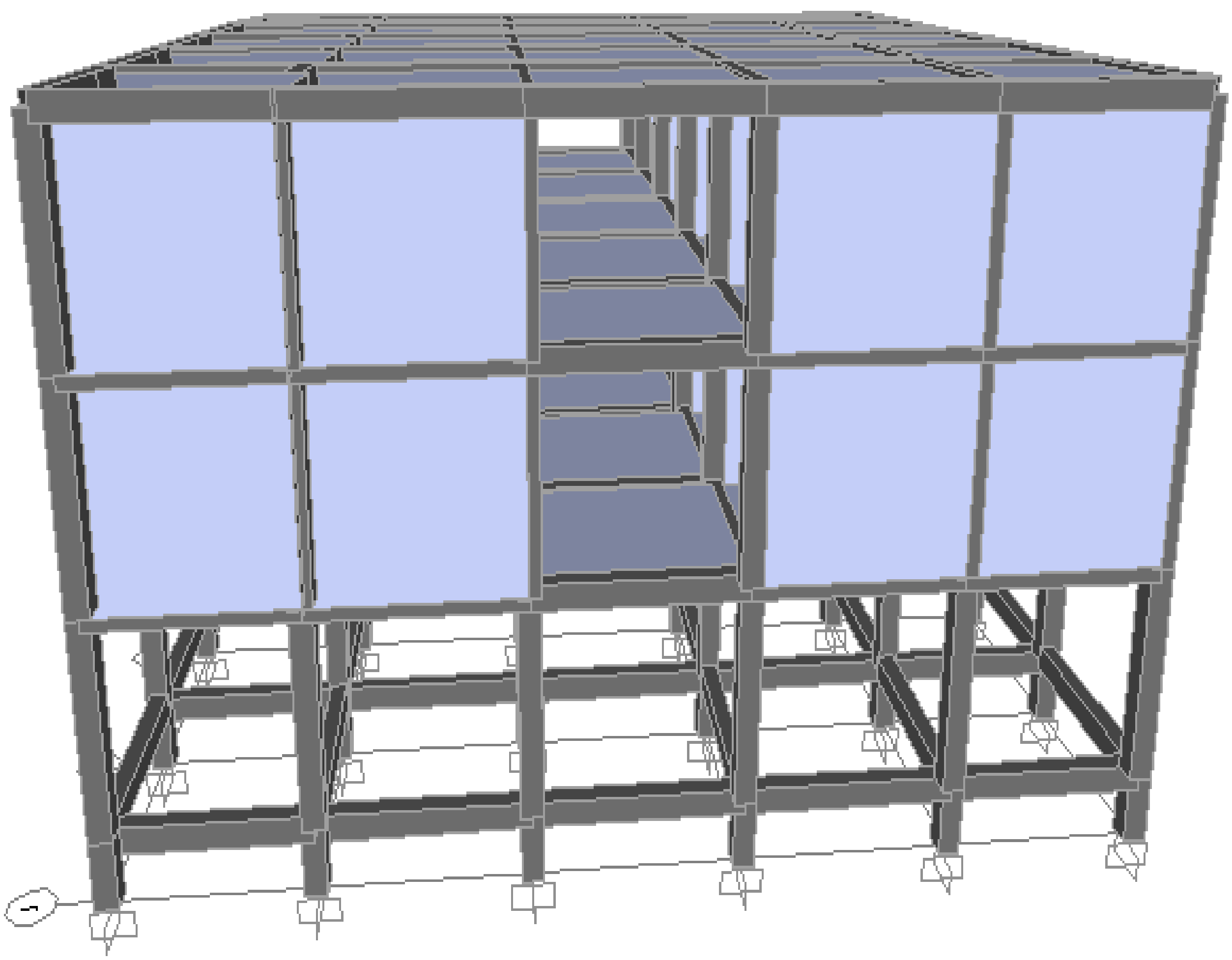


Figure 1 Typical Multi-Storey Building

1.2 Problem Statement:

Incorporation of galvanized welded steel wire mesh is an efficient way to provide improved seismic performance. Welded wire mesh location in the structure is quite important in the structure. So to see whether its placement in the structure at potential plastic hinge region will improve the ductility or not, a thorough analysis and study is required.

1.3 Work Procedure:

To analyze the seismic performance of the concrete structures by addition of galvanized steel welded wire mesh the following steps are carried out.

- Selection of a 3-storey building
- Structural Modeling of selected building on SAP2000
- Gravity analysis of a building
- Non-linear static Pushover analysis of a building in SAP2000
- Casting and testing of concrete cylinders with and without welded wire mesh as confinement to reinforcement
- Development of moment curvature (M-Phi) curve from stress-strain curve of samples tested
- Modifying the hinge properties in SAP2000 by inputting moment curvature curve
- Non-linear static pushover analysis of a model in SAP2000
- Compilation of results
- Conclusions and recommendations

1.4 Aims & Objectives:

The focal aims of this research work are as follows;

- Study the seismic response of RC structure and compare it with and without using galvanized steel welded wire mesh in the potential plastic hinge region.
- This study will tell the improvement in ductility and hence the seismic response of buildings.
- This study will also help the professionals to add welded wire mesh in the beam-column joint region in the design to improve seismic performance.

The main objectives of this research work are as follow;

- Developing a basic model of the building on SAP2000
- Gravity analysis of RC structure
- Non-linear static analysis of the model with and without adding the welded wire mesh properties in the potential plastic hinge region

1.5 Utilization:

It can be used in the construction of commercial buildings, plazas and multi-storey residential buildings especially in earthquake prone areas. It is also very economical and easily available in the market. It will enhance the confinement and hence the seismic behavior of the buildings to a significant level. It can be implemented throughout Pakistan and especially in areas lying in seismic zone 3 such as KPK, FATA, PUNJAB and Federal territory of Pakistan.

2.Literature Review

2.1 Introduction:

Nowadays, with the catastrophic activity and safety measures Engineers, researchers and general public come to a same page of evaluating a building under seismic responses in order to minimize the losses due to earthquake. Demand from the public view is much more as compare to technical managements teams. This is due to the fact that a lot of work on seismic performance improvement of the buildings and structures. No proper analysis is performed while designing of the new buildings. So it results in the lot of damage caused as a result of seismic activity. In this regard in the foreign countries a lot of study has been going on in the field of earthquake. Conclusions of some research papers are listed below which will help us in carrying out our project.

2.2 Seismic Response of Building Frames with Vertical Structural Irregularities

CONCLUSIONS:

After studying this research paper:

- The evaluation of 3 and 5 stories of frame RC structure has been made and earthquake response of that structure has been studied.
- five storey building was evaluated which was in 2-D
- The response of time history analysis was compared therewith using ELF using UBC code standards
- Based on this comparison, the aim was to check whether ELF procedure was applicable or not.

2.3 Effects of plastic hinge properties in nonlinear analysis of reinforced concrete buildings:

CONCLUSIONS:

The following conclusions were observed:

- Base shear capacity of RC structure with default property of plastic hinge and the modified user hinge for various lengths of plastic hinge is evaluated
- Transverse reinforcement spacing are identical but the variation within the base shear capacity is a smaller than 5%.
- Thus, the base shear capability doesn't rely upon whether the modified property of default property of hinge is used. Plastic hinge region has a drastic effect on lateral displacement of a structure.
- Our research shows that with the modification of plastic hinge region 30% increment in displacement is observed
- Lateral displacement depends on the quantity of transverse reinforcement at the potential hinge regions used.
- Comparisons shows that transverse reinforcement obviously increase the capacity at plastic hinge region
- At yielding, both the default as well as user define hinge is activated and absorb energy as well
- There are interesting variation at ultimate state in plastic hinge region. Although, the location of hinge is same the model with default hinges emphasizes a ductile beam mechanism with weaker beam mechanism; damage or failure happens at the beams.

2.4 A review of research on seismic behaviour of irregular building structures since 2002:

CONCLUSIONS:

- The research on earthquake structure has been date back to nineteenth century. Large research has been carried out sine to improve the concrete structure response especially within the nonresilient range mainly of RC building models.

- Different techniques has been studied to improve the response under seismic loading. These devices will certainly reduce the torsional response. On the other hand, Major unstable codes for vertically irregular concrete buildings have resulted in satisfactory unstable performances.

- Some research has been devoted for improvement in nonlinear response of RC structure achieving good result with dynamic analysis.

- In conclusion, very little research is still in pushover analysis as revealed by little number of paper on vertical irregularity of buildings.

2.5 Practical Three Dimensional Nonlinear Static Pushover Analysis:

CONCLUSIONS:

- Gravity pushover analysis is forced control procedure while lateral pushover is displacement control. SAP2000 permits the distribution of lateral force employed in the pushover to be supported the same acceleration in an exceedingly specific direction, a specific mode form, or a user-defined static load case.

- Building is simply stiffening and run the analysis again simply give the results
- Assumptions are the change using modify hinge property criteria.

2.6 Nonlinear Static Pushover Analysis of an Eight Story RC Frame-Shear Wall Building in Saudi Arabia:

Conclusions:

After this research the result shows that Madinah Municipality building is deficient of lateral load more specifically earthquake loading. Most of the member start to yield as small lateral load or push is applied. Plastic hinges will form on members. Nonlinear or ductile behavior due to plastic hinges is represented by pushover curve or hysteresis loops. These loops are then compared using backbone curves. Enhancement in lateral load is done using modification. Shear wall is one technique which takes the lateral load and strengthen the building. Based on response spectrum performance points are are 0.094m and 0.097m.

2.7 Seismic Evaluation of Reinforced Concrete Frames Using Pushover Analysis

For analyzing the seismic response of structure nonlinear static analysis is done to check the vulnerability of structure under lateral load. Nonlinear analysis shows that structure design uder gravity load has some capacity to resist earthquake load. Main Points include of conclusion include:-

1. Plastic hinge first form in bottom storey beams and then failure transfer to columns. This clearly shows the strong column weak beam phnomenon

2. Plastic hinge brittle behavior puts building under large risk level under seismic loading. Trend should be shift toward ductile behavior.

3. Structural Modeling and Analysis

3.1 Selection of Building:

There are different types of buildings which have been built and there are many others which are still under construction in Pakistan. Islamabad being the capital of Pakistan contains many commercial, residential and institutional buildings which high rise buildings are.

We have selected the 3 storey building for our analysis which is similar to typical buildings which are being constructed in Pakistan. Our building has the gallery or open space in the middle portion. We want to study the earthquake performance of the RC structure and hence to suggest improvement in weak beam-column joint location for improved seismic response. So analysis of building is necessary.

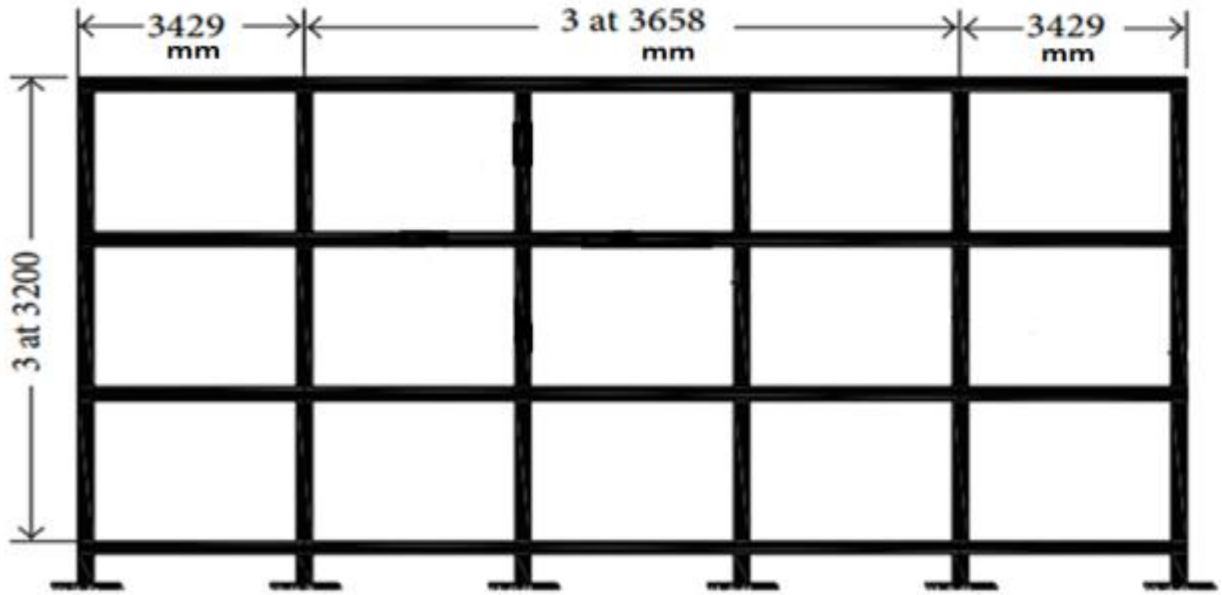


Figure 2 Elevation of Selected Frame

3.1.1 Model Characteristics:

Size of Model	60’x60’
No. of Stories	3
Storey Height	10.50’
No. of Bays in X-Direction	5
No. of Bays in Y-Direction	5
Beam Size	12”x15”
Column Size	15”x15”
External Wall Thickness	9”
Reinforcement	#9 main bars, #4 stirrups
Cover	2.5”
Longitudinal Spacing	6”

3.2 Gravity Analysis:

By means of gravity analysis it means that analyzing the structure on the basis of gravity loads only. Before performing the seismic or pushover analysis the structure must pass in gravity test. Passing of the structure under gravity loading means that the structure must be safe within the allowable limits under gravity loading.

By gravity loading it is meant that load acting due to force of gravity that is the vertical forces. Gravity load includes the weight of the structure itself, human and other things occupancy load and snow load imposed on the structure. All these loads should have a complete load transfer path to the ground. Engineered structures are composed of multiple types of structural members that are connected to each other to transfer the load to the bottom of the structure. Load should be transfer from top roof to bottom foundations safely. Load path should be safe, stiff and continuous. Every member should resist load upon it and rest on support. It is designed to support the gravity load. The whole load path is explained below:

1. First of all the load travels from the floor slab to the beams on which it is supported.
2. Upon reaching the ends of the beam load travels from the ends of the beam to the girder below on which it rests.
3. The girder supports the load coming from the slab and beam lying above it and transfers this load to the connecting columns lying beneath it.
4. The load then travels from through the column down to the foundation from where it is distributed to the ground lying beneath it.

3.2.1 Development of basic Model:

Our work was on SAP2000. First step was to build the 3D model on SAP which was carried out in the following way:

The first step was to define the grids. By defining grid we mean that assigning coordinates or positions. The initial number of grids in X,Y and Z direction were 5,6 and 7 respectively having spacing between them of 20’,18’ and 12’ respectively. We modified these grids according to our own model requirements.

After that the next step was to define materials which were given in building characteristics. We defined additional material named masonry because it was listed in already defined materials list. We gave it properties of bricks like elastic modulus and other properties. Next step was to define the frame sections which were defined arbitrarily (checked later on). Frame sections consists of beams and columns. The building consists of same columns throughout. These two were defined under rectangular frame sections option. The section properties and all other modification factors were kept to default values. The concrete reinforcements were added which were to be checked later on and therefore reinforcement to be designed option was marked. For this purpose #9 main reinforcement bars were provided and #3 transverse bars were provided. The cover for the main bars were 2.5” and longitudinal spacing given was 6”.

Proceeding further we defined the area sections which were slab and wall. The slab was designed as thin shell and wall was defined as membrane .The slab thickness was kept as 6” and the wall thickness was kept as 9”. The remaining values were set to default.

The next step after defining area sections was to assign the loads to the structure. We assigned live load of 60 psf. Then we assigned the load combination of 1.2 X Dead load+1.6 X Live load.

3.3 Pushover Analysis:

3.3.1 Inelastic Methods of Analysis:

Inelastic analytical procedures have to be there for analysis of the structures since these behave inelastically when earthquakes strike. Failure mode are analyzed with help of inelastic analysis and pregressive damage is also observed. In Inelastic behavior time history analysis can be done on real time earthquake data. Dynamic response is sensitive to model ground motion chracterisitcs. There is a need of correct modeling of ground shake to analyze correctly.But a real time history curves are needed for that purposes which in most cases are not available or expensive. Because of its simplicity nonlinear analysis is a good technique to evaluate building. Static analysis counter nonlinear property of a material. It uses a spectrum technique. Displacement control method is use using FEMA 273 and secant method is used.

Displacement-based procedures offer a additional rational approach to those problems relative to force control procedures using inelastic deformations method instead of elastic forces. The tool for analysis method ought to even be comparatively easy which may capture important response parameters that considerably have an effect on the analysis method.

3.3.2 Description of Pushover Analysis:

In pushover analysis model is artificially apply with the lateral load pushes in both directions till a displacement reached at which it fails occur reaches. Analysis gives the displacement, stresses and moment at every point and also the graph can also be computed using software. First a building is analyze under gravity loading using its own self weight then a predefined lateral pushes are applied in order to study the stiffness of structure under lateral loading. This method is displacement control and displacement of top storey is measured. The roof displacement is premeditated with base shear to induce the world capability curve. Nonlinear staic analysis is a well known technique to check the seismic capacity of structure by the foremost rehabilitation tips as a result of it's conceptually and computationally easy. In this process sequence of yielding on every members and structural failure of that member due to its capacity is studied. It finds the demand of a structure to be compare with its capacity. It identify the weak joints or more specifically weak links and thus help in techniques to improve that region. It expose the weakness of structure beyond elastic range.

3.3.3 Types of Pushover Analysis:

Pushover analysis is of two sorts. It will either be performed as force-controlled or displacement-controlled. Force-controlled is employed once the load is understood (such as gravity loading). In displacement-controlled procedure is employed wherever the magnitude of applied load isn't best-known before. The load is accrued till the known displacement reaches a specified worth. Generally, roof displacement at the middle of mass of structure is chosen because the control displacement. the inner forces and deformations at the target displacement provide inelastic strength and deformation demands that is compared with obtainable capacities to seek out a performance point. during this study displacement based procedure is employed for unstable loads and forced based procedure for gravity loads.

3.3.4 Limitations of Pushover Analysis:

Although pushover analysis has benefits over elastic analysis procedures, but the assumptions for pushover analysis and limitations of current pushover procedures should be known. The estimate of target displacement, choice of lateral load patterns and identification of failure mechanisms because of higher modes of vibration are vital problems that have an effect on the accuracy of pushover results.

In pushover analysis, the target displacement of a MDOF system is calculable because the displacement demand for the corresponding equivalent SDOF system. A form vector representing the deflected shape of the MDOF system is employed to get the properties of constant SDOF system. a set form vector, elastic initial mode, is employed for simplicity while not considering the upper mode effects by standard approaches.

The distribution of inertia forces vary with the severity of earthquake and with time throughout earthquake since but, in pushover analysis, usually an invariant lateral load pattern is employed. The lateral load patterns employed in pushover analysis are proportional to product of story mass and displacement related to a form vector at the story into consideration. normally used lateral force patterns are uniform, elastic initial mode, “code" distributions and one targeted horizontal force at the highest of structure. The invariant lateral load patterns couldn't predict potential failure modes because of middle or higher story mechanisms caused by higher mode effects. Invariant load patterns will give adequate predictions if higher mode effects aren't vital.

These limitations have led several researchers to propose adjustive load patterns that take into account the changes in inertia forces with the level of physical property. though some improved predictions are obtained from adjustive load patterns, they create pushover analysis computationally hard and conceptually sophisticated.

3.3.5 Pushover Analysis Using SAP2000:

In our study we have done Pushover Analysis regardless of its limitations because it was easy to perform and interpretation of results was simple.

We have performed the Pushover Analysis of our building model in the following way:

- ❑ Linear Static Analysis of our Reference Model/Building using Seismic Loading
- ❑ Linear Static Analysis of modified Model after experimentation (Compression testing).
- ❑ Backbone curve comparison of base and modified model.

Hinge Color	Description
Pink	Initial Occupancy
Blue	Life Safety
Light Blue	Collapse Prevention

Hinge Assignments Auto (From Tables in ASCE)	
Hinge Type	
M2M3 for beams	PM2M3 for columns
Hinge Overwrites	Auto (Subdivide Line Objects)0.02

Formation of hinges is shown in the results. The results show that collapse hinges are developing in columns at basement storey level, which is undesirable. All other hinges are developing in the beams, so basement story column should be strengthened. Then the building under consideration is strengthened by providing shear walls at different locations. Pushover analysis is run using same load cases and results are viewed.

The base model (having default hinge properties) was compared with the modified model by incorporating the effect of GW mesh after having extracted the moment curvature curve from the stress strain diagram of tested samples. New moment and curvature capacity values were input in the user defined hinge properties and again the pushover analysis was run in order to obtain the backbone curve.

4. Results and Discussion:

Hysteresis loops obtained for the base model and the modified model were used to make the backbone curves by joining the peaks and later comparison was done to study the change in base shear and displacement capacity of the 3-storey building.

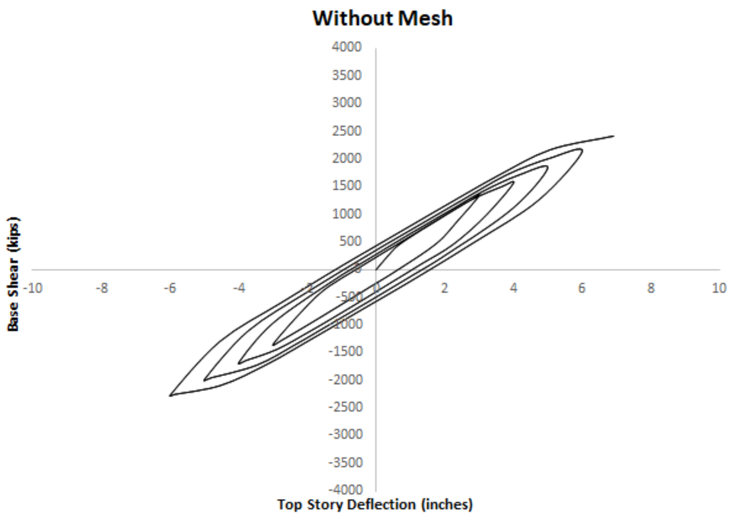


Figure 3 Hystersis Loop of Structure without Mesh

The above figure shows the hysteresis loop which is obtained after performing the pushover analysis. This is the loop of bas

e model. Our base model failed at 9th Push with the displacement of 7.1 inches.

5 Experimentation

5.1 Introduction:

3 standard samples of 6” diameter and 12” height were casted. One sample of PCC, one RCC sample with 4 #1 bars as main reinforcement and #1 bars used as lateral reinforcement with a spacing of 3” and one RCC sample with Galvanized Welded Wire Mesh (GW Mesh) incorporated in it.

5.2 Materials :

5.2.1 Galvanized Steel Welded Wire Mesh (GW Mesh):

Locally available GW mesh was used as an addition confinement. Figure 1 below shows the mesh used in this experimentation. It was fabricated using a 0.7 x 1.25 Grade 40 wire which was welded in 16mm grid pattern. As Kusuma at el has shown that welded wire fabric provides much better concrete confinement than rebar reinforcement system, GW mesh is expected to show the same result.

Figure 4 Locally available galvanized Steel Welded Wire mesh (GW Mesh)

5.2.2 Steel:

In both the samples of RCC, 3.175mm Grade 60 steel was used as longitudinal and lateral reinforcement.

Figure 5 Reinforcement without GW Mesh

Figure 6 Reinforcement with GW Mesh

5.2.3 Concrete:

M20 grade concrete (compressive strength of 20MPa) was used with a mix design of 1 : 1.5 : 3(cement: sand: gravel) by weight. This mix design was selected keeping in view the compressive strength typically used in Pakistan for multistory buildings.

5.3 Instrumentation:

Compression tests were performed after 28 days of curing using a 2000 kN capacity Compression testing machine ASTM C39 with a loading rate of 0.15 MPa /sec. A digital extensometer can be seen in the figure to measure the axial deformation. Figure 4 shows the compression test setup and specimen undergoing the test. Progress of the test was monitored on the computer screen and all the load-deformation data was stored.

Figure 7 Specimen ready for testing with full set up

Compression Testing Results:

Figure 8 shows the stress strain curve of all the 3 tested samples. It can be observed that the behavior of all the samples is almost the same and linear till the elastic limit (proportional limit) is reached. It can be seen that RCC sample without mesh has more strain ductility than the PCC sample. When comparing the RCC sample without mesh with the meshed sample, a considerable increase in the ductility can be observed. Strain taken by PCC was 0.00192 whereas a difference of 0.0006 can be seen amid with and without meshed samples, which is an increase of 14.29% in the strain due to the incorporation of GW mesh. There is also an increase in the strength of the sample having mesh due to an increased confinement provided by the mesh.

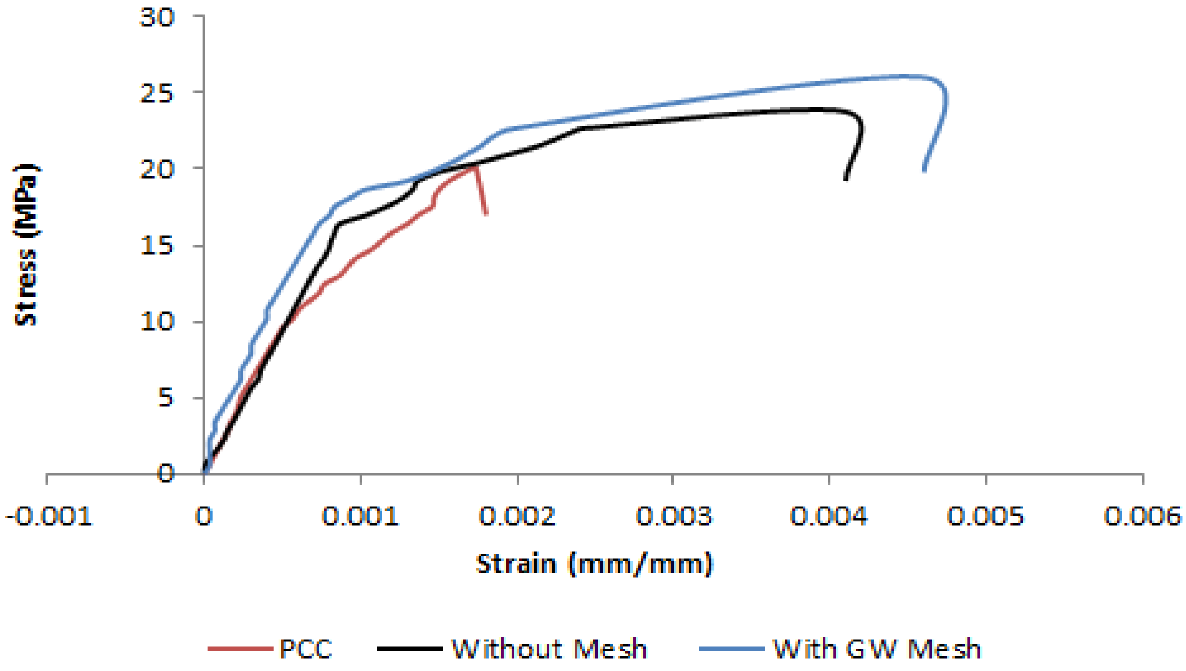


Figure 8 Stress- Strain Curve Comparison

Moment Curvature Curve

Initial modulus of elasticity and secant modulus of elasticity were extracted from the stress-strain curve for the formation of moment curvature curve. The material properties are summarized. The moment-curvature curve, which represents the concrete ductility, has four inflection points :

- o The concrete decompression
- o The cracking of tensioned concrete
- o The yield of tension steel rebars or/ and of the compressed concrete
- o The collapse of the section, reached by crushing of compressed concrete or by breaking of tensioned steel rebars

Figures 9, 10 and 11 on the next page show the input parameters, output parameters and the associated curves of PCC, Meshed and without Mesh samples. The combined M-phi curve of the samples with and without mesh have been shown in the figure depicting a considerable increase in the curvature capacity at yielding as well as at failure ie . Yielding curvature and ultimate curvature observed in the mesh confined model is 0.00016005 and 0.000556946 respectively which is 6.52% and 10.23% more than that the model without mesh. An increase of 6.55% in curvature at cracking is seen to occur in the meshed model.

PCC

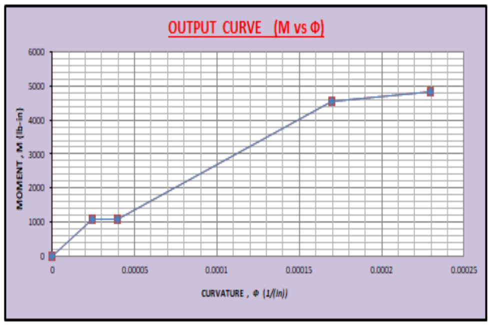
EXCEL SHEET FOR MOMENT CURVATURE RESPONSE

INPUT PARAMETERS

MATERIAL PROPERTIES		DIMENSION PROPERTIES	
f'c (psi	4000	b (in)	15
fy (psi)	60000	h (in)	25
η	16.59158972	d (in)	22.5625
ft (psi	474.341649	Cover (in)	1.5
Ec (ksi	1747	db (in)	1.125
Ey (ksi	28985.50725	ds (in)	0.375
Esec(ksi	1367.35	No. of Bars	4
		As (sq.in)	3.9760875

OUTPUT PARAMETERS

Curvature (Φ)	Moment (M)
0	0
2.45218E-05	1067.136611
3.97099E-05	1067.136611
0.00016967	4559.599003
0.000229381	4824.654154



CALCULATION AREA

BEFORE CRACKING					AT CRACKING & YIELDING						BEYOND YIELDING					
Centroid (y)	Moment Of Inertia (uncracked) (I)	Distance b/w Centroid and Centre of Section	Cracking Moment(Mcr)	Curvature (Φ)	kd	I cracked	Cracking Moment(Mcr)	Curvature (Φcr)	Moment (at yielding)	Curvature (Φ)	β1	a	Centroid (Can)	Tensile Strain (es)	Ultimate Nominal Mome	Curvature (Φ)
13.93	24910.03325	11.0724964	1067.1366	2.45218E-05	10.36010027	15382.55936	1067.136611	3.97099E-05	4559.599003	0.00016967	.85	4.678	5.503	0.009	4825	0.000229381

Figure 9 Excel Sheet for Moment Curvature Response PCC

4" Dia without mesh

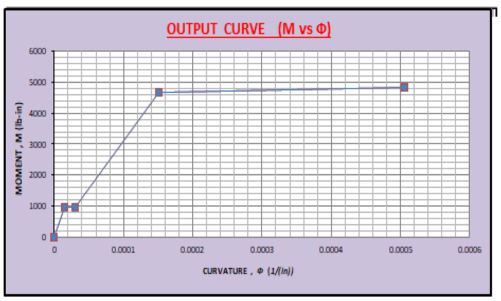
EXCEL SHEET FOR MOMENT CURVATURE RESPONSE

INPUT PARAMETERS

MATERIAL PROPERTIES		DIMENSION PROPERTIES	
f'c (psi	4000	b (in)	15
fy (psi)	60000	h (in)	25
η	10.5632315	d (in)	22.5625
ft (psi	474.341649	Cover (in)	1.5
Ec (ksi	2744	db (in)	1.125
Ey (ksi	28985.50725	ds (in)	0.375
Esec(ksi	840.42	No. of Bars	4
		As (sq.in)	3.9760875

OUTPUT PARAMETERS

Curvature (Φ)	Moment (M)
0	0
1.49361E-05	943.4313676
3.02591E-05	943.4313676
0.000150258	4684.802587
0.000505242	4824.654154



CALCULATION AREA

BEFORE CRACKING					AT CRACKING & YIELDING						BEYOND YIELDING					
Centroid (y)	Moment Of Inertia (uncracked) (I)	Distance b/w Centroid and Centre of Section	Cracking Moment(Mcr)	Curvature (Φ	kd	I cracked	Cracking Moment(Mcr)	Curvature (Φcr)	Moment (at yielding)	Curvature (Φ)	β1	a	Centroid (Can)	Tensile Strain (es)	Ultimate Nominal Mome	Curvature (Φ)
13.43	23019.08885	11.5736162	943.43137	1.4936E-05	8.784067131	11362.41407	943.4313676	3.02591E-05	4684.802587	0.000150258	.85	4.678	5.503	0.009	4825	0.000505242

Figure 10 Excel Sheet for Moment Curvature Response without mesh

4" Dia with mesh

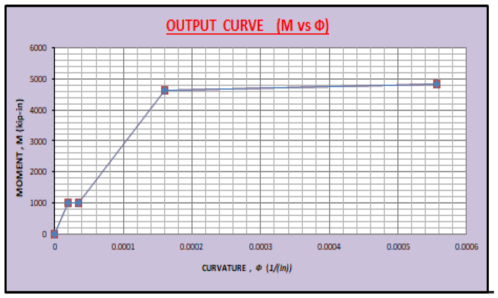
EXCEL SHEET FOR MOMENT CURVATURE RESPONSE

INPUT PARAMETERS

MATERIAL PROPERTIES		DIMENSION PROPERTIES	
f'c (psi	4000	b (in)	15
fy (psi)	60000	h (in)	25
η	13.51305699	d (in)	22.5625
ft (psi	474.341649	Cover (in)	1.5
Ec (ksi	2145	db (in)	1.125
Ey (ksi	28985.50725	ds (in)	0.375
Esec(ksi	644	No. of Bars	4
		As (sq.in)	3.9760875

OUTPUT PARAMETERS

Curvature (Φ)	Moment (M)
0	0
1.95329E-05	1004.338991
3.48085E-05	1004.338991
0.000160047	4617.870492
0.000556946	4824.654154



CALCULATION AREA

BEFORE CRACKING					AT CRACKING & YIELDING						BEYOND YIELDING					
Centroid (y)	Moment Of Inertia (uncracked) (I)	Distance b/w Centroid and Centre of Section	Cracking Moment(Mcr)	Curvature (Φ	kd	Icracked	Cracking Moment(Mcr)	Curvature (Φcr)	Moment (at yielding)	Curvature (Φ)	β1	a	Centroid (Can)	Tensile Strain (es)	Ultimate Nominal Mome	Curvature (Φ)
13.68	23971.03979	11.3213394	1004.339	1.9533E-05	9.626592525	13451.40246	1004.338991	3.48085E-05	4617.870492	0.000160047	.85	4.678	5.503	0.009	4825	0.000556946

Figure 11 Excel Sheet for Moment Curvature Response with mesh

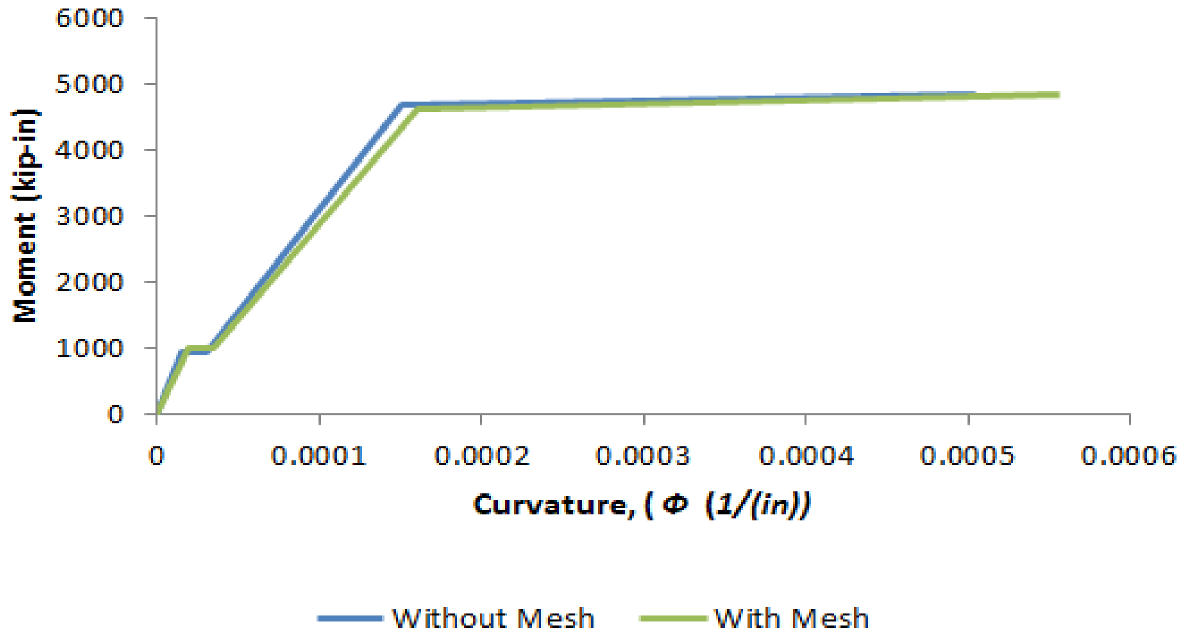
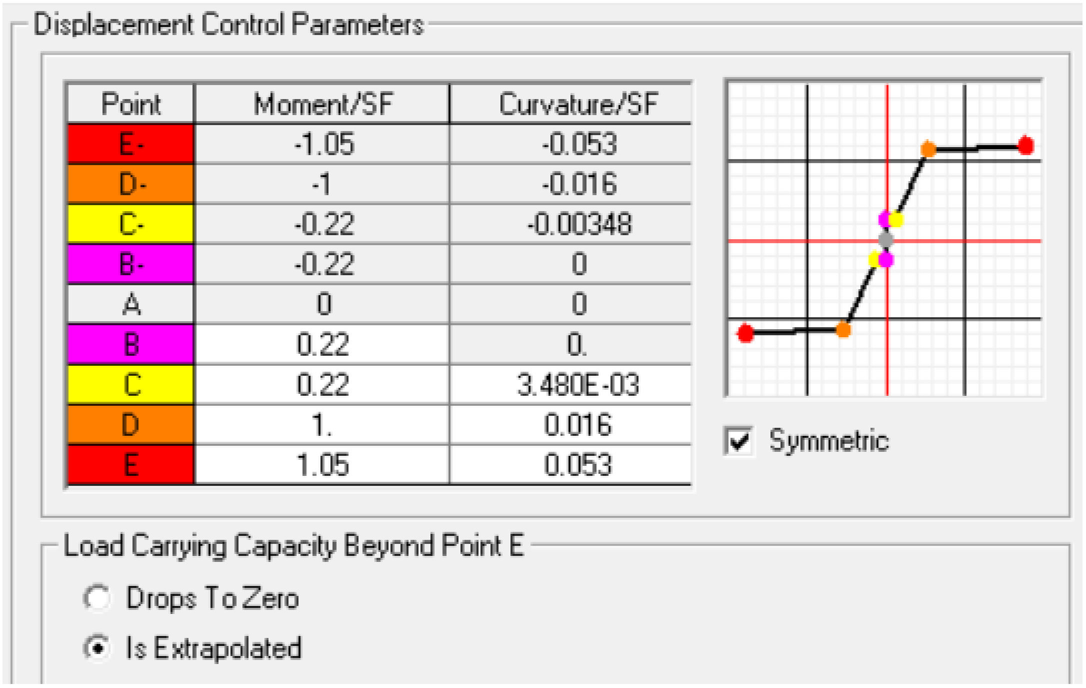


Figure 12 Moment Curvature Curves Comparison

Static non linear Pushover Analysis with user defined hinge properties:



Point	Moment/SF	Curvature/SF
E-	-1.05	-0.053
D-	-1	-0.016
C-	-0.22	-0.00348
B-	-0.22	0
A	0	0
B	0.22	0.
C	0.22	3.480E-03
D	1.	0.016
E	1.05	0.053

Figure 13 Displacement Control Parameters

We modified the plastic hinge properties by inputting the moment curvature curve values in displacement controlled parameters. The points B,C,D and E above are the occupancy levels. So by doing this our hinges were modified and then with these modified plastic hinge properties again non linear static pushover analysis is performed in SAP2000.

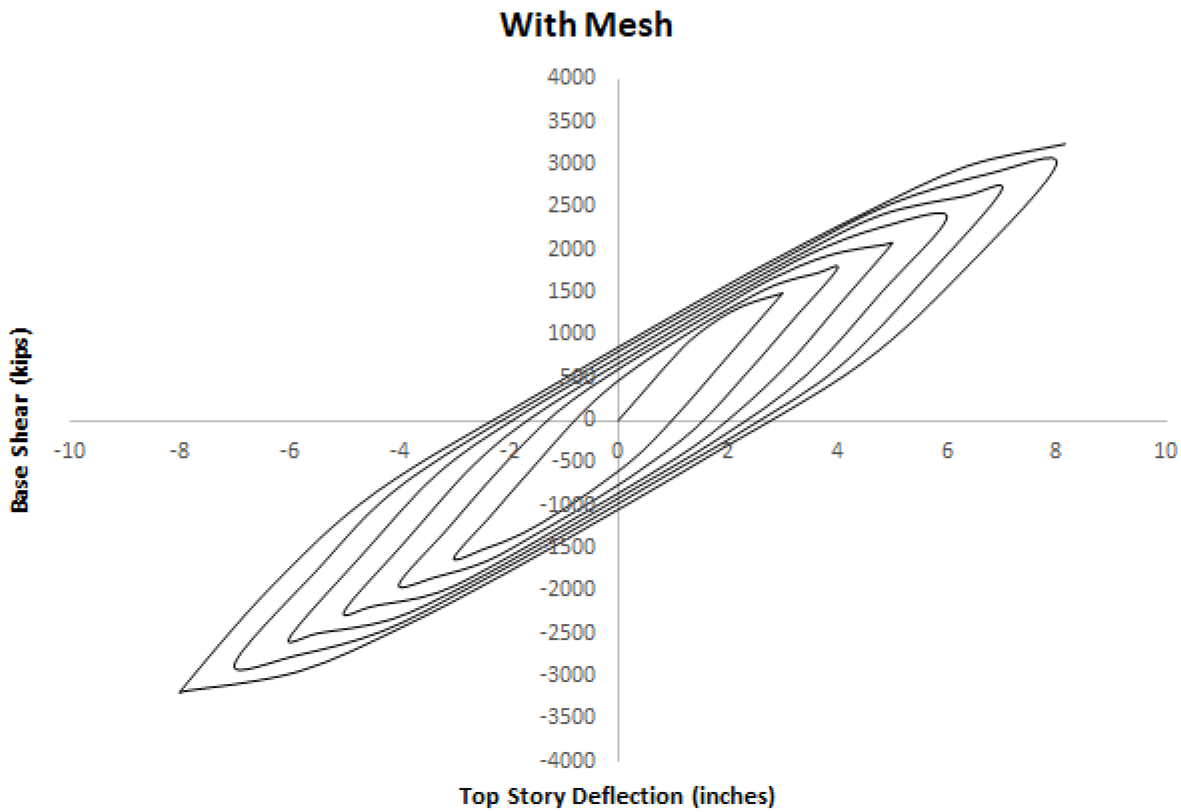


Figure 14 Hysteresis Loop of Structure with Modification

Figure 14 shows the hysteresis loop of the modified model after incorporation of modified hinge properties. The above loop shows that our structure failed at 13th push having the displacement of 8.2 inches.

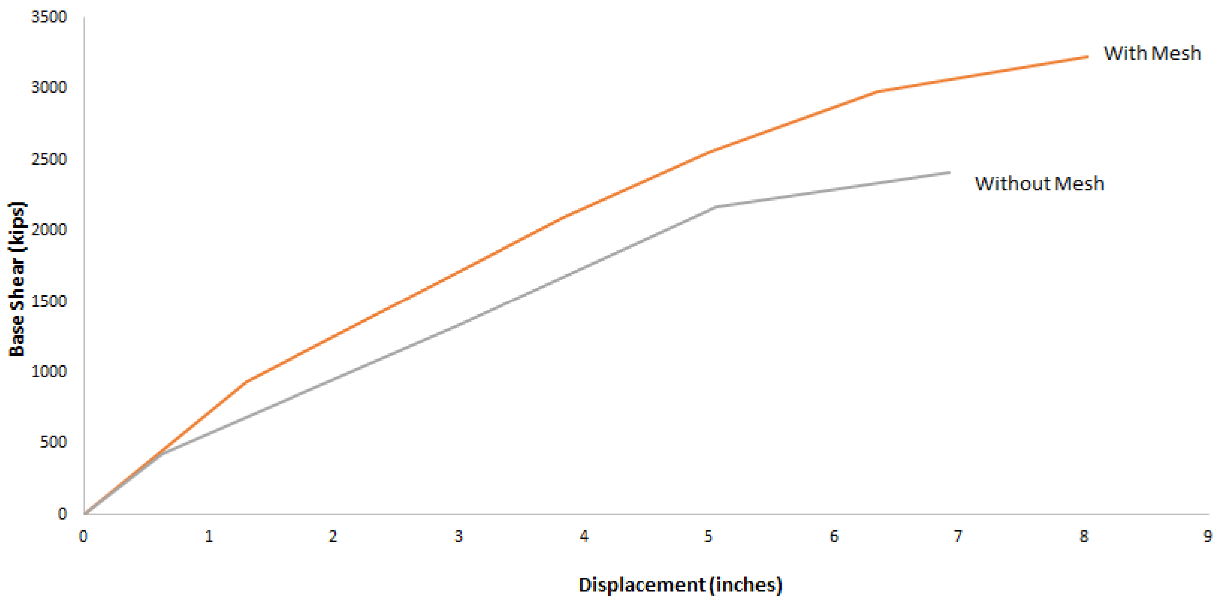


Figure 15 Back-bone Curves Comparison

Figure 15 is the backbone curve of the modified model which is obtained after joining the peak values of each individual curve of hysteresis loop. The backbone curve clearly indicates the increase in curvature by 16.91% with the addition of Galvanized welded steel wire mesh which was required.

6 Conclusions

1. Galvanized welded Steel wire mesh can be used as a confinement to concrete core along with conventional ties.
2. GW mesh increases the strain of concrete specimen by 14.29% with respect to conventional lateral confinement only.
3. There are lesser cracks in the concrete core in the meshed sample as compared to the sample without mesh due to uniform confining stress redistribution.
4. The strength of the concrete is relatively decreased at the expense of the increased ductility but it is almost not considerable.
5. There is an increase of 10.23% in the ultimate curvature due to insertion of mesh at almost same moment.
6. There is an increase of 9.29 % in the strain ductility of the material
7. There is an increase of 5.77 % in the curvature ductility of the section
8. Top storey deflection of the model is increased by 15.49 %
9. Base shear of the model is increased by 29. 88 %
10. An increase of 1.2 inches in displacement and around 700 kips increase in base shear is observed for 3-storey building in this research by incorporating mesh in the plastic hinge regions of beams and columns of soft storey.

7 References

I. Valmundsson, E.V. and Nau, J.M., 1997. Seismic response of building frames with vertical structural irregularities. Journal of Structural Engineering, 123(1), pp.30-41.

II. Inel, M. and Ozmen, H.B., 2006. Effects of plastic hinge properties in nonlinear analysis of reinforced concrete buildings. Engineering structures, 28(11), pp.1494-1502.

III. De Stefano, M. and Pintucchi, B., 2008. A review of research on seismic behaviour of irregular building structures since 2002. Bulletin of Earthquake Engineering, 6(2), pp.285-308.

IV. Ashraf Habibullah, S.E. and Stephen Pyle, S.E., 1988. Practical three dimensional nonlinear static pushover analysis. Structure Magazine, 2.

V. Salahuddin, H., Ali Khan, S., Usman Ali, M. and Mushtaq, A., 2017. Seismic vulnerability assessment of CFRP strengthened RC structures. Građevinar, 69(02.), pp.121-130.

VI. Guevara-Perez, L.T., 2012. “Soft Story” and “Weak Story” in Earthquake Resistant Design: A Multidisciplinary Approach. In Proceedings of 15th World Conference on Earthquake Engineering (pp. 24-28).

VII. Kusuma, B. and Suprobo, P., 2011. Axial Load Behavior of Concrete Columns with Welded Wire Fabric as Transverse Reinforcement. Procedia Engineering, 14, pp.2039-2047.

VIII. Mander, J.B., Priestley, M.J. and Park, R., 1988. Theoretical stress-strain model for confined concrete. Journal of structural engineering, 114(8), pp.1804-1826.

IX. Soltani, M., An, X. and Maekawa, K., 2004. Cracking response and local stress characteristics of RC membrane elements reinforced with welded wire mesh. Cement and concrete Composites, 26(4), pp.389-404.

X. Xiong, G.J., Wu, X.Y., Li, F.F. and Yan, Z., 2011. Load carrying capacity and ductility of circular concrete columns confined by ferrocement including steel bars. Construction and Building Materials, 25(5), pp.2263-2268

XI. .Xiong, G.J., Wu, X.Y., Li, F.F. and Yan, Z., 2011. Load carrying capacity and ductility of circular concrete columns confined by ferrocement including steel bars. Construction and Building Materials, 25(5), pp.2263-2268.

XII. Inel, M. and Ozmen, H.B., 2006. Effects of plastic hinge properties in nonlinear analysis of reinforced concrete buildings. Engineering structures, 28(11), pp.1494-1502.

XIII. Ashraf Habibullah, S.E. and Stephen Pyle, S.E., 1988. Practical three dimensional nonlinear static pushover analysis. Structure Magazine, 2.

XIV. Tabsh, S.W., 2007. Stress-strain model for high-strength concrete confined by welded wire fabric. Journal of materials in civil engineering, 19(4), pp.286-294.

XV. Antony, R.R. and Pillai, P.S., EFFECT OF VERTICAL IRREGULARITIES ON SEISMIC PERFORMANCE OF RC BUILDINGS.

XVI. Shahzada, K., Gencturk, B., Khan, A.N., Naseer, A., Javed, M. and Fahad, M., 2011. Vulnerability assessment of typical buildings in Pakistan. International Journal of Earth Sciences and Engineering, 4(6), pp.208-211.

XVII. Ali, M.U., Khan, S.A., Anwar, M.Y. and Gabriel, H.F., 2015. Probabilistic application in seismic vulnerability assessment of deficient low-to medium-rise reinforced concrete buildings in Pakistan. Arabian Journal for Science and Engineering, 40(9), pp.2479-2486.

XVIII. Maqsood, S.T. and Schwarz, J., 2008. Seismic vulnerability of existing building stock in Pakistan. In Proceedings of the 14th World Conference on Earthquake Engineering, Beijing, China. Paper (No. 09-01, p. 122).