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Contents

Analysis and Behaviour the Concrete Columns Strengthening with the Carbon Polymer Fibres

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A B S T R A C T

In recent years using the polymer materials in strengthening and retrofitting the concrete elements is on the developing process. Fiber-reinforced polymers (FRP), have emerged as an alternative solution to traditional materials for strengthening and retrofitting structures, especially in existing structures which need to up build or to change the destinations. In attempt to increase strength and ductility of reinforced concrete (RC) load bearing elements through confining systems the FRP membranes have become a familiar solution. Extensive studies (experimental, finite element modeling and analytical modeling) were carried out on the analysis of confining effect in case of concentrically loaded RC columns. This paper investigates the prospect of strengthening deficient square columns and cycle columns with carbon fiber-reinforced polymer(CFRP) jackets. In both cases output results will compare with the etalon specimen (without strengthening). Currently, the study of RC columns confined with composite materials will be focused in centric compression, because the eccentric compression is relatively new and limited. FRP confinement systems are less effective under eccentric loading compared to concentric. Experimental program on testing the performance of centric loaded RC columns externally strengthened with CFRP membranes was carried out and results are presented in this paper.

Keywords Column, Concrete, Ductility, Fibres, Strengthening, Confinement

1. INTRODUCTION

Many studies concerned with strengthening of existing reinforced concrete columns, especially in seismic regions and in changed the destination of structure, have focused on providing additional confinement to the core concrete elements by means of external reinforcement. From the studies that have been conducted over the past several years, the advantages of using FRP materials have become more apparent. Strengthening can be achieved in a column by wrapping the column with different cross sections using fiber reinforced polymer.

Due to the increase in the ultimate compressive strain, the ductility capacity and energy absorption capacity are also considerably improved, always in comparing the different sections of columns. In all different case studies the results are compared with “etalon” without strengthening.

Using and analyzing the experimental output data, our aim is to verify the hypothesis:

- effect of fully wrapping the concrete columns
- possibility effect of partially wrapped of concrete columns
- the effect of section the concrete columns , advantages and disadvantages

Because in recent years, external CFRP systems have become widespread in field column applications despite only limited experimental research data on the seismic response of CFRP wrapped specimens Different types of strengthening and different sections result with optimizations and orientations toward the benefits. The methodology of applying the CFRP, such wrapping was in accordance with technical specifications of materials. (L.C.Hollaway,M.K. Chryssanthopoulos and S.S.J.Moy (2004))

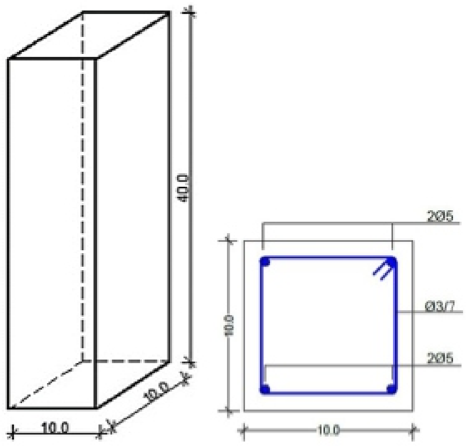

Figure 1a. Rectangular cross section

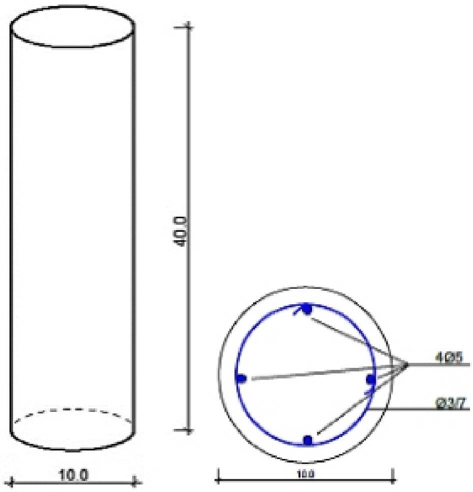

Figure1b. Circular cross section

2. TEST SETUP AND TESTING PROCEDURES OF SAMPLES

The columns were axially loaded using a displacement control compression apparatus in the laboratory.The three different types of samples are prepared for two types of columns: Circular and Rectangular cross section in previous approved model, presenting in fig 1a and 1.b.The rectangular cross section is analysing in two types of wrapping: fully and partially wrapped. For each set of the specimens are used three samples.

2.1. Materials Properties

The concrete mix design for all the specimens is approved according to the EN-206-1, presented in table 1. The concrete mix design is based on the dimension of the models.

Table 1. Materials and properties of materials

Material	Mass(kg/m^3)	Concrete properties
Cement/CEM II 42.5	340	Compressive Strength(fc'=38.5)N/mm^2
water	180	Slump=150 mm
Coarse aggregate	780	W/C =0.53
Fine aggregate	1080	Max.Aggregate size-16 mm
additives	1.5	Air entrante 3.5 %

The CFRP, using for strengthening are from MAPEI-Italy, and properties are presented in table 2

Table 2. Materials and properties of CFRP

Description	MAPEWRAP C QUADRI-AX 760/48
Mass (gr/m2)	760
Density(gr/cm3)	1.79
Thickness (mm)	0.106
Tensile strength (N/mm2)	>4800
Tensile modulus of Elasticity (GPa)	230
Elongation (%)	2.1

2.2. Test Specimens

To determine the effect of CFRP wraps on column strength, 3 circular and 3rectangular columns were manufactured and tested (Katarina GAJDOSOVA, Juraj BILCIK). The details of the column specimens are shown in Figures 1a and 1b. The properties and dimensions of three circular columns were selected to be similar in order to obtain a reasonable mean value of the results. The rectangular column specimens were analyze in two different strengthening (wrapped) ways: full wrapped and partially wrapped (like this stirrups). The analyzing specimens were wrapped using one layer of CFRP, but in analytical analyzing are presented also for more layers, is it requested. All the geometrical and mechanical properties of CFRP are presented in table 2.

3. TESTING PROCEDURE

The columns were axially loaded using a displacement control compression apparatus in the laboratory. The first column (circular– Set D) was loaded with a speed so that the displacement increased until failure. Displacement and ultimate strength were recorded throughout the entire tests. The mode of failure for each specimen was presented in Figure 2.

The rectangular columns (two types-of wrapped -Type A and B) are also tested in same way and in same conditions.

The plain (etalon-non strengthening) specimens in both cases (Set C-rectangular and set E –circular) are tested using the same procedures.

4. ANALYZING TESTING RESULTS OBSERVATION

4.1. Circle Specimens

Specimen D: Failure was sudden. It was observed through a crack which formed approximately 150mm from the top of the columns. The CFRP appeared to have burst due to the pressure or straincaused by the load.

Figure 2. Test set up and shape of failure of circular specimens

Figure 3. Test set up and shape of failure of rectangular(fully wrapped) specimens

Figure 4. Test set up and shape of failure of rectangular (partially wrapped) specimens

4.2. Square Specimens(fully wrapped)

Specimen A (fully wrapped): The columns showed warning signs of failure and cracked approximately middle of the column but in one side through edge . There were visual signals of failure such as particle debris , but not only in layer of CFRP.

4.3. Rectangular Specimens (partially wrapped)

Specimen B (partially wrapped): The columns showed warning signs of failure and cracked approximately in middle of the column but in all of surfaces . There were visual signals of failure such as totally debris, in concrete structure.(fig 4)

5. ANALYZING AND COMPARING THE RESULTS

To be able to determine the confining effect of CFRP wrap on the behaviour of reinforced concrete columns, the axial compressive strength of concrete columns without CFRP wrap are calculated.

$$N_{Rcc,d} = \frac{1}{\gamma_{Rd}} A_c \cdot f_{ccd} + A_{sl} \cdot f_{yd} \qquad (1)$$

A_c -concrete cross section area; A_s - area of longitudinal reinforcement f_{ccd} -strength of strengthening concrete

In our case study we used the different cross section, and the factor of cross section is presented in formula (3)

$$k_{\alpha} = \frac{1}{1+(\tan\alpha)^2} = \frac{1}{1(\tan 90)^2} = 0 \qquad (2)$$

The calculations in our case is based also in the tensile strength of wrapped materials, presented in formula (3)

$$ff_{ffff,w}(RR) = ff_{ffffff} + \langle \eta\eta_{RR} ff_{ffff} - ff_{ffffff} \rangle \qquad (3)$$

and effective stress in our type of wrapped , presented in formula (4)

$$\sigma\sigma_{ffffff} \leq 0.004EE_{ff} \qquad (4)$$

Analytical calculations for different cross sections specimens is presented in table 3.

Table 3. Analytical Calculated strength for different cross sections/after strengthening

Cross section	rectangulare	cicrle
Axial Strength $N_{Rcc,d}$ (kN)	**175.68**	**418.00**
Shear Strength $V_{Rd,f}$ (kN)	**14.02**	

The analysis of results after examinations present very wider spread results, especially in compare the results in rectangular columns were the indicate factor is very slightly in improvement of bearing capacity under axial loads. In general the results are more under of our aim for rectangular columns , but in circle columns the results are more than our aim in beginning Comparing the results of all tested specimens , including also the plain samples is presented in table 4.

Table 4. Compare the all tested sample for different cross sections

Set	sample	Date of preparation	Date of testing	age	Cross section	Dim A x b(cm)	Height (cm)	Mass (gr)	Type of wrapped	Compression force (kN)	Compressive strength (N/mm²)
A	MM_1	31.03.14	07.05.14	28	Rectang.	10x10	40	9531	Fully wrapped	331.81	33.18
	MM_2	31.03.14	07.05.14	28	Rectang.	10x10	40	9508	Fully wrapped	361.20	36.12
	MM_3	31.03.14	07.05.14	28	Rectang.	10x10	40	9541	Fully wrapped	368.29	36.83
B	MM_1	31.03.14	07.05.14	28	Rectang.	10x10	40	9492	Partially/ stirrups	318.14	31.81
	MM_2	31.03.14	07.05.14	28	Rectang.	10x10	40	9502	Partially/ stirrups	328.35	32.84
	MM_3	31.03.14	07.05.14	28	Rectang.	10x10	40	9520	Partially/ stirrups	369.91	36.99
C	MM_1	31.03.14	07.05.14	28	Rectang.	10x10	40	9484	No wrapped	261.75	26.18
	MM_2	31.03.14	07.05.14	28	Rectang.	10x10	40	9519	No wrapped	288.61	28.86
	MM_3	31.03.14	07.05.14	28	Rectang.	10x10	40	9564	No wrapped	278.45	27.85
D	MM_1	31.03.14	07.05.14	28	circle	d=10	40	8295	Fully wrapped	555.13	70.72
	MM_2	31.03.14	07.05.14	28	circle	d=10	40	8320	Fully wrapped	590.81	75.26
	MM_3	31.03.14	07.05.14	28	circle	d=10	40	8332	Fully wrapped	518.11	66.00
E	MM_1	31.03.14	07.05.14	28	circle	d=10	40	8342	No wrapped	221.58	28.23
	MM_2	31.03.14	07.05.14	28	circle	d=10	40	8320	No wrapped	231.25	29.46
	MM_3	31.03.14	07.05.14	28	circle	d=10	40	8344	No wrapped	236.13	30.08

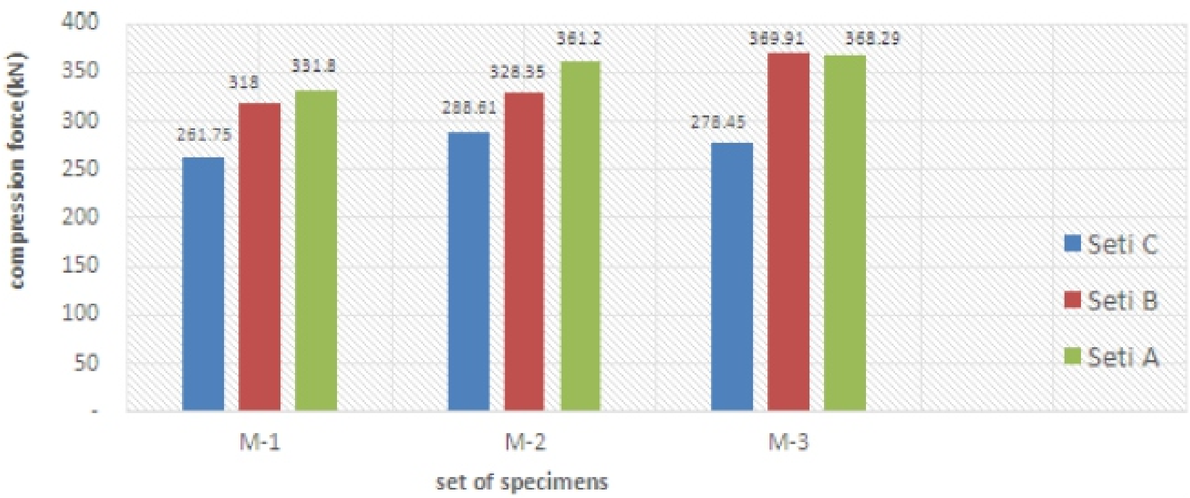

Figure 5. Comparison the different sets of rectangular columns

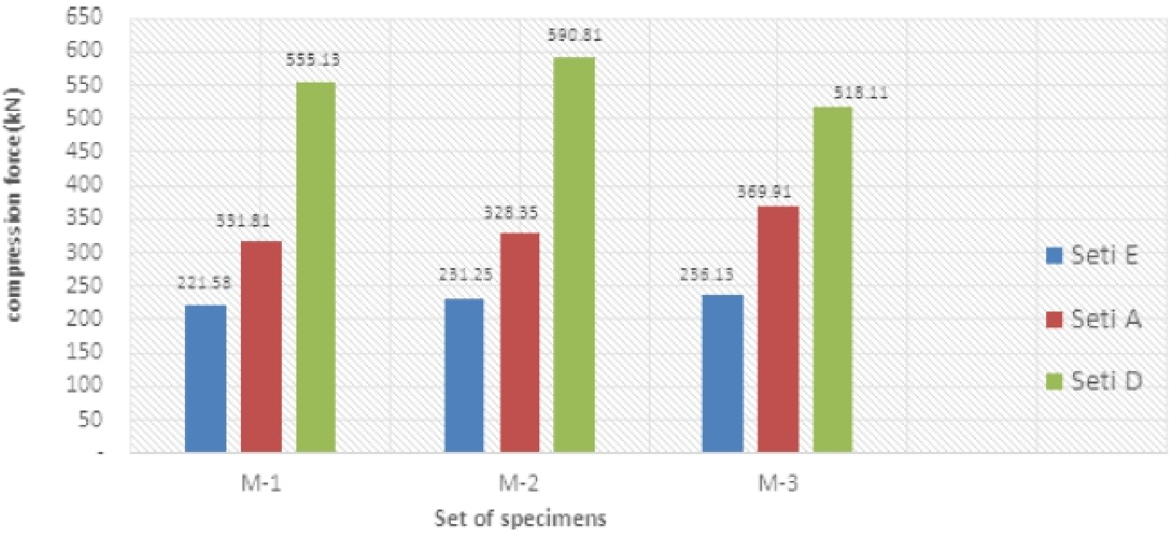

Figure 6. Comparison the different sets of circle columns

The analyze of all cases in this paper is presented in graphical form, using the charts in fig 5 and fig.6.

Comparison between the test results of wrapped rectangular columns with the calculated value express not significant changes between set A and B, and very slightly increasing compare with set A (no strengthening), presented in fig. However, if the corners of the square columns are rounded appropriately, the axial strength and ductility of columns increase considerably.

Comparison between the test results of wrapped circular columns shows that the CFRP wrap can increase the axial strength of circular columns significantly. The ductility of circular columns improves because of the application of the CFRP wrap. Comparing with set A, is significant increase the axial loads. (presented in fig. 6)

6. CONCLUSIONS

The effect of the CFRP wrap on the axial strength of reinforced concrete columns was analyzed in this paper. The study included testing two types' columns in three series.

The first series comprised three similar rectangular columns fully wrapped with CFRP (set A). The second series consisted of three similar rectangular columns partially wrapped with CFRP (Set B). The third series consisted of three similar circle columns fully wrapped with CFRP (set D). Based on the test results, the following conclusions are drawn:

- Applied the strengthening of circle columns is indicated factor on increasing of axial loads and in same time creating the more ductile element.
- The applied the strengthening fully wrapped and partially wrapped is not big different, because the main indicated factor is rounded the edges, and applications in this case is limited.
- The partially wrapped it will be more applicable in positions were the shear force have the main values.
- The recent proposed equations presented for the wrapped circular columns can be used to predict the axial strength of CFRP wrapped circular columns. However, their proposed equations for square columns overestimate the axial strength of CFRP wrapped square columns unless the corners of the square columns are rounded appropriately.

- The idea of the work was to analyze the improvement the bearing capacity of the columns under axial loads , using the preferable method , wrapped with CFRP, and the results are satisfy our aim for circle columns but not for rectangular columns.
- The main problem in rectangular columns is the edges in sections, was the stresses are concentrated, and already is the weak point.

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A Critical Appraisal of Off-land Structures: A Futuristic Perspective

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A B S T R A C T

There will be crisis of land leading to the need of development of the infrastructure for residential, commercial, industrial & agricultural use due to exponential growth in population, current and projected. The metropolitan cities are developing at a very high rate and the expecting rise in population is putting pressure on these cities to grow further by expanding their boundaries continuously. But in the case of cities like Mumbai, Chennai and international coastal cities, the sea is behaves like a boundary for urban settlement, resisting its further expansion. This situation has produced challenges for urban planners to deal with lack of space and demand for basic amenities. In such condition coastal metropolitans, oceans can be used to develop the floating form of satellite towns, urban pockets & structures as a coastal expansion of city boundaries due to the enormous flexibility and limitless possibilities which water offers. The objective of this paper is to highlight the possibilities of off land development on water. It begins with the review of need of floating structures and their scope towards development & growth of cities. It includes a brief history and case studies related to evolution and development of floating structures from our past to present and finally the authors presents different types of design methodology with which these ideas can be implemented.

Keywords Population Explosion, Land Shortage, Off-land Structures, Coastal Metropolitan

1. INTRODUCTION

The earth contains all kind of resources to sustain life and need of all human beings, plants & animals. But it is greed of humans which is exploiting these resources to fulfill their endless greed. According to the medium variant of the 2010 Revision of World Population Prospects as shown in Figure 1 the world population is expected to increase from 6.9 billion in mid-2011 to 9.3 billion in 2050 and to reach 10.1 billion by 2100. [1].

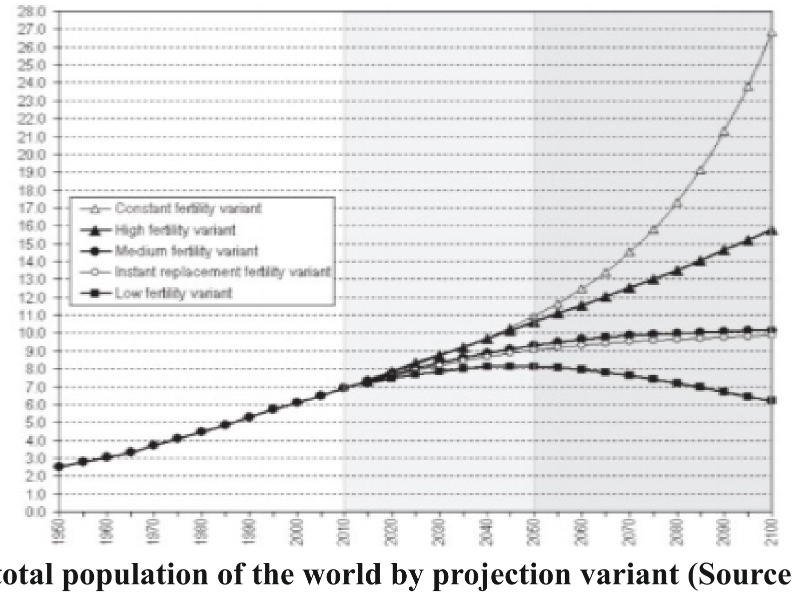

Figure 1. The total population of the world by projection variant (Source:www.un.org)

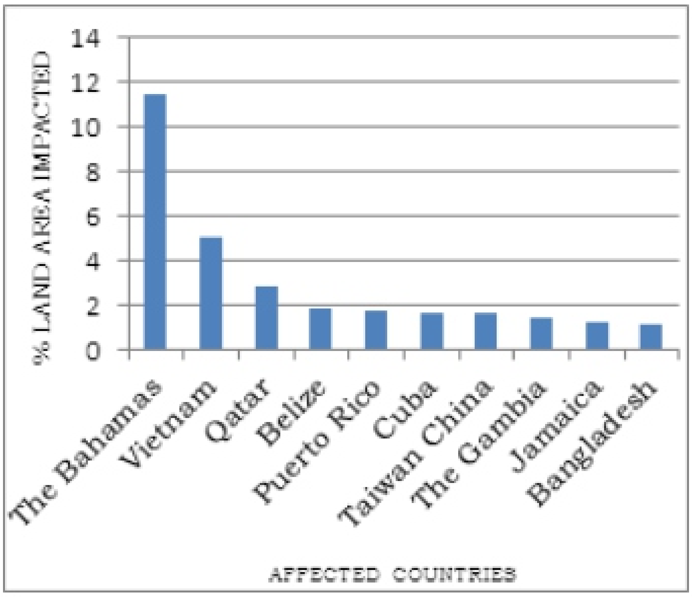

Figure 2. The percentage land area of the most impacted countries (Source: www.ibtimes.com)

The majority of the population growth in upcoming years will occur in urban areas of developing countries. The basic and the most important resource for the survival of human being is the land. For a long term perspective for our future land will become a major issue. According to US Geological survey, presently we have land area of about 148,300,000 sq. km, as about 29% of total surface area of earth, from which almost 50% is unsuitable for habitation & agricultural purpose due to poles, deserts & hard rocks & mountains. the remaining 71% surface area of planet earth is occupied by water [2], & from this total quantity, 96.54% founds in Oceans, Seas, & Bays [3], which is about 68.54 of total surface area According to forecasts of the IPCC, the ocean level will rise from 20 to 90 cm during the 21st century (versus 10 cm in the 20th century) [4]. According to the research, published in the Proceedings of the National Academy of Sciences, a global mean temperature elevation of 1oC will lead to rise of 2 meter in mean sea level [5]. Increase of 1 m would bring out the world to loose of approximately 0.31% of ground & 1, 30% of GDP. Further effect of this rise of 1M in MSL is given in Table 1 and Figure 2 shows the most impacted countries according to land area. [6]. The objective of this paper is to study the possibilities of off-land structures on water. It begins with the need of floating structures and their scope of contribution towards development and growth of cities. It includes a brief history and case studies related to evolution and development of floating structures and finally the authors conclude with the different design methods for off-land structures.

Therefore we need to find out proper solutions for upcoming problems due to population explosion, increase in sea level & limited amount of available land. For agricultural and industrial purpose we have very limited usable land area (approximate 12% of earth) so development of off land structure is the option where we can look forward for solution of our problems related to housing, employment and infrastructure and investment. This development of ‘off land’ structure in sea will definitely reduce the load on the shoulders of our mother earth.

The development of such kind of structure will be a new concept initially in present time, but within a very short span of time in future this will become the necessity and trend.

2. RESEARCH METHODOLOGY

In this paper qualitative research methodology is used. A critical analysis of the possibilities of off-land structures on water has been done. The paper also discusses the need of floating structures and their scope for the growth of urban centers. Further the paper also validates the study by three case studies of off-land structures namely The Greenstar and the Floating Golf Course at Maldives and a hypothetical study of a Floating Ecopolis. An appraisal of the different design methods and design considerations of off-land structures has also been done.

Table 1. Summary of world regional impacts due to rise in sea level (Source: www.openknowledge.worldbank.org)

Indicators	World	Latin America & Caribbean	Middle east & North Africa	Sub Saharan Africa	East Asia	South Asia
	1 M SLR					
Area	0.31	0.34	0.25	0.12	0.52	0.29
Population	1.28	0.57	3.2	0.45	1.97	0.45
GDP	1.3	0.54	1.49	0.23	2.09	0.55
Urban Extent	1.02	0.61	1.94	0.39	1.71	0.33
Ag. Extent	0.39	0.33	1.15	0.04	0.83	0.11
Wetlands	1.86	1.35	3.32	1.11	2.67	1.59
	5M SLR					
Area	1.21	1.24	0.63	0.48	2.3	1.65
Population	5.57	2.69	7.49	2.38	8.63	3.02
GDP	6.05	2.38	3.91	1.42	10.2	2.85
Urban Extent	4.68	3.03	4.94	2.24	8.99	2.72
Ag. Extent	2.1	1.76	3.23	0.38	4.19	1.16
Wetlands	7.3	6.57	7.09	4.7	9.57	7.94

3. HISTORICAL DEVELOPMENT OF OFF-LAND STRUCTURES

The development of off-land structures is not a new concept for the world. Early applications of floating structures take the form of floating boat bridges over rivers that date back to antiquity. Figure.3 shows About 480 BC, King Xerxes of Persia led his army across the Hellespont, now called the Dardanelles, using two rows of floating bridges, each consisting of about 300 boats laid side by side. [7]. Such kind of floating bridges were mainly built as a necessity of war, permitting the movement permitting the movement of soldiers and equipment between ground bases and marine vessels during that period. Later on many bridges were constructed on this concept of tying the boats & ships. Other than the bridges there are many examples of very old floating villages throughout history. They have appeared in different parts of the world due to various needs and reasons.

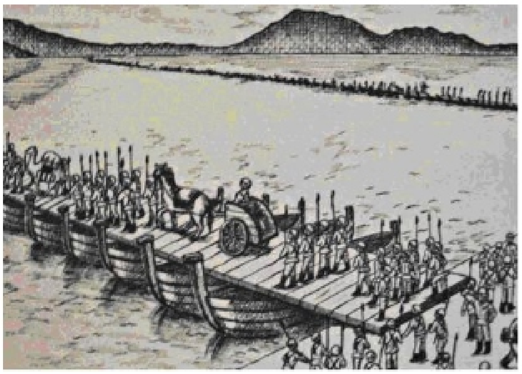

Figure 3. The floating boat bridge by King Xerxes of Persia (Source: Hammond and Roseman, 1996)

Figure 4 shows in the sixteenth or seventeenth century, Ganvie's floating city (Sometimes called the Venice of Africa) was established by the Tofinu people, as a protective measure from Dahomeys which was a slave trading tribe that dominated the region during the time. [8] Other than this example, Vietnam, Indonesia. Thailand. China. Peru and Bolivia also have a history of floating villages. There are many countries which have floating habitation not for permanent living but for the tourism purpose, like In India, boat houses in Kerala and Kashmir are important attraction of tourists.

Figure 4. Ganvie's floating city, Africa (Source: www.afritorial.com)

In our contemporary history use of floating structures other than bridges, primarily starts after industrial revolution. Original concept of floating habitation begins by the primitive man who built small boats as a means of navigation. Later on, as man started progressing he built bigger boats, which were used to cross the sea. It took him several days, months. He had to stay on these boats and this gave rise to the concept of boat houses [9]. With the development in construction and technology men designed huge ships (cruises) as warships and as a mode of transport that can accommodate more than thousands of people.

Numerous proposals have been made for very large floating structures. Proposed applications ranges from the visionary floating city to more likely floating airport, military bases, wave power generators and deep ocean mining platforms. Till now, most of the off land buildings and structures have been developed for oil exploration and tourism only but due to future scenario these all are going to become a need with mix of technological advancement. Advancement in term of technologies can be of construction, development of new materials, energy generation system, security and control system.

4. SCOPE OF OFF-LAND STRUCTURES

The scope of neighborhood sized, self-sustained independent 'off land' floating structure can be decided on basis of the criteria of land use systems of different categories like agriculture and industries should be promoted to be used on ground. These kinds of manufacturing and production based structures have heavy concentrated load and don't need to be used in luxurious way, so we need to reduce maintenance, operational and transportation cost related to production. The purpose to build the 'off land' structure is to provide an isolated, peaceful and efficient environment for working and living. For this kind of structure, preferably residential, office, institutional, academic, commercial or recreational buildings are needed to be developed as single or mixed use form. For example, IT parks with all basic amenities and residential features for its employees can be developed in the form of floating structures. Residential townships can be developed to cater the requirements of farm houses, beach houses, & bungalows. This kind of specific block will be self-sustained in itself.

Academic buildings like colleges and universities can also be developed in form of off land structure. This category required bigger area in peaceful environment so these kinds of structures can be accommodated in such urban pockets. Jails and prisons can be design in such structures in isolated forms from main town. In such case it will be very easy to handle safety and security based issues. Yoga and Meditation centers also need isolated and peaceful environment, so these structures can be designed here in form of floating structures. Entertainment facilities & utility plants are also one of the important categories of buildings which can be designed in floating form. Emergency rescue bases in floating form can be developed in earthquake prone areas like Japan, (sea sided earthquake prone countries) and in flood prone areas. These off land structures ought to be designed as net zero energy buildings as well as with net zero carbon foot print and self-sustained in terms of energy by using renewable energy resources to make it environmentally favorable. Economically-the development cost will be very high so it will target a specific category higher income group.

5. CASE STUDIES

There are various off land structures have been constructed and few are under construction in the form of floating structure. Few prominent examples have been summarized as below:

5.1. Green Star, Maldives

A floating hotel and conference center named as Greenstar has been designed by Water Studio, Netherlands (Figure 5) in Maldives (a nation widely considered to be on the front lines of global warming because of its sea level). The Greenstar, contains 800 rooms and a Conference Center for up to 2000 participants. The Greenstar will blend-in naturally with the existing surrounding islands. The green covered star-shape building symbolizes Maldivians innovative route to conquer climate change. This will become the next location for conferences about climate change, water management and sustainability. A unique Floating Restaurant Island will be built next to it [10].

Figure 5. The Greenstar, floating hotel and conference center, Maldives (Source: www.waterstudio.nl)

5.2. Floating golf course, Maldives

Another important floating structure going to be developed in Maldives is the world's first floating golf course. Threatened with rising sea levels from climate change, the island nation may be doomed to a watery grave unless it transitions to floating developments. Developed by Dutch Docklands and designed through a collaboration between golf course developer Troon Golf and Waterstudio.NL, the zero-footprint solar-powered golf course will be one of the first floating developments and is expected to bring in a wave of new tourists. it is also designed to minimize its impact on the surrounding ecosystem. The artificial floating islands will incorporate technologies such as water cooling, water desalination, and the use of floating solar blanket fields. Underwater tunnels will connect the holes and facilities together, while allowing golfers to experience the reef. [11].

5.3. Lily Pad - The Floating Ecopolis

In 2008 a conceptual model of The Floating Ecopolis (Figure 6a & Figure 6b), known as the Lilypad, was designed by Belgian architect Vincent Callebaut. He says, "A true Amphibian, half aquatic and half terrestrial city, able to accommodate 50,000 inhabitants and inviting biodiversity". Callebaut imagines his structure at 250 times the scale of a lilypad, with a skin made of polyester fibres coated in titanium dioxide which would react with ultraviolet light and absorb atmospheric pollution.

Figure 6(a). The Lilypad (Source: Vincentcallebaut architectures)

Figure 6(b). The Bird's eye view of Lilypad ((Source: Vincentcallebaut architectures)

The Lilypad comprises of three marinas and three mountain regions with streets and structures strewn with foliage. "The goal is to create a harmonious coexistence of humans and nature," said Callebaut. With a central fresh water lagoon acting as ballast, the whole construction would be carbon neutral utilizing solar, thermal, wind, hydraulic, tidal and osmotic energies. With high density populations living in low-lying areas -- The Netherlands, Polynesia, Bangladesh -- the Ecopolis, its creator believes, could be the answer to mass human displacement that global warming is predicted to cause [12].

6. DESIGN METHODS FOR OFF LAND STRUCTURES

The major issue for designing such off-land structures is related to transportation and connectivity. In case of transportation, huge energy is consumed. So, design of such mega floating structure should be in such a manner to minimize the horizontal travelling distance. A clustered form of planning in the form of vertical towers can be good for decreasing the transportation distances. This condition will be defined by concentrated load bearing condition of these structures. There should be proper means to have access to boat, ship, submarine & helicopter. Another important issue will be energy production and consumption by these off-land structures. The most important renewable sources for onsite energy production for sustainable off land structure are solar, wind and hydroelectric energy. Using onsite renewable energy, these off land structures may play an important role in reducing our carbon footprint.

At site level condition, a breakwater (usually needed if the significant wave height is greater than 4 m as a thumb rule) is needed for reducing wave forces impacting the floating structure (Figure 7). This breakwater will be formed by a ring of columns around the structure with the option to bear the turbines to generate the electricity due to movement of waves [13]. The mooring system (Figure 8) must be well designed so that it can ensure the stability of very large floating structure to keep it in position so that the facilities installed on the floating structure can be reliably operated and to prevent the structure from drifting away under critical sea conditions and storms.

A freely drifting very large floating structure may lead to damage not only the surrounding facilities but also the loss of human life if it collides with ships. Note that there are a number of mooring systems such as the dolphin-guide frame system, mooring by cable and chain, tension leg method and pier/quay wall method (Figure 8). Another feature could be the design of the edges of the structure. By proper choice of edge layout, the propagation of the incident waves into the main part of the structure can be reduced by efficient scattering or reflection of the incident waves on the weather side [13]

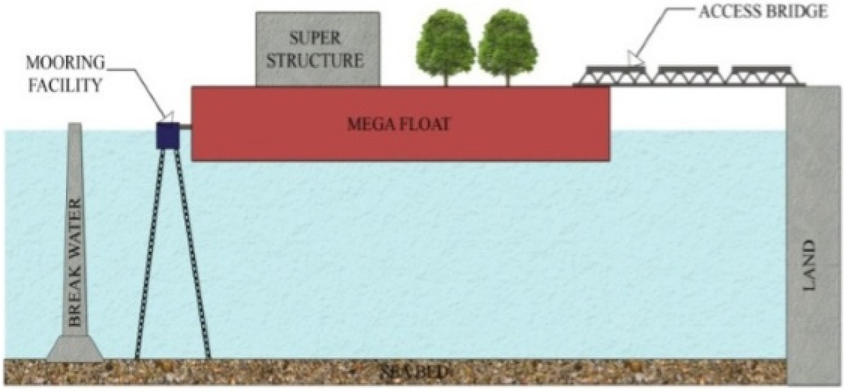

Figure 7. Components of off land structures

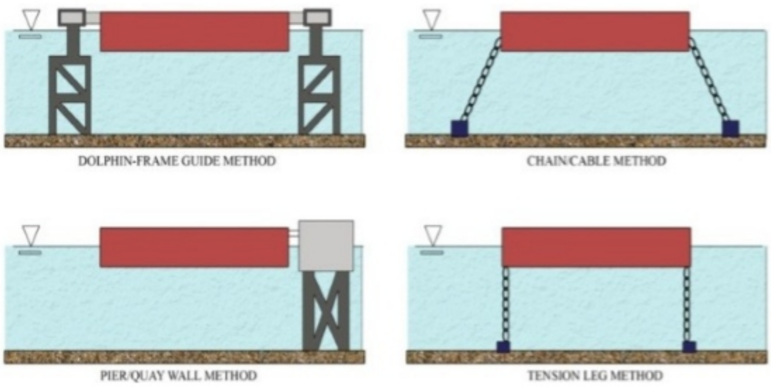

Figure 8. Different types of mooring systems.

The materials to be used for the floating body may be steel, or concrete or steel-concrete composite and the relevant specifications have to be followed for the main structure. This concrete will be in category of special water tight, light weight concrete to increase water tightness of structure and to decrease overall density by reducing its dead load. In case of the buildings of the structures we will use recyclable material having lesser energy consumption according to their life cycle assessment, to make such structure environment friendly [13].

7. DESIGN CONSIDERATIONS FOR OFF LAND STRUCTURES

Adequate performance of off-shore structures has to be ensured by designing them to comply with serviceability and safety requirements for a service life of 100 years or more. Serviceability criteria are introduced to ensure that the structure should fulfill the function required to perform as, and are specified by the standards, owner and user. Major serviceability requirements are related to performance during movement of waves. An important design issue regarding safety of personnel is evacuation and rescue. An effective safety measure related to this can be to provide a safe place where people can survive or board after anaccident. In principle the global failure modes of floating structures include capsizing, sinking, structural failure and drift-off. For such condition we have to design the installed blocks on main floating surface as they should also be floating type independently in itself. This will be a safety measure if there is any disaster due to any reason if main structure tends to submerge in water then these structures will start flowing independently. Planning wise it can be designed in form of radial planning form with a control center & main building as a nucleus. Other buildings can be planned in a concentric ring from to be placed as per decreasing priority form center to periphery.

Technically there will be some other considerable load based factor in design of floating structures such as abnormal loads like impact loads due to collision of ships with the floating structure, effects of earthquakes including dynamic water pressure, effects of temperature change, effects of water current, effects of tidal change, effects of seabed movement, snow load, effects of tsunamis, effects of storm surges, ship waves, seaquake, brake load, erection load, effects of drift ice and ice pressure, effects of drifting bodies, effects of marine growths etc. These mega-floats will be protected from seismic shocks since they will be inherently base isolated. They will not suffer from differential settlement due to reclaimed soil consolidation. As these urban pockets will also be very high in cost so they will have higher surface coverage and Floor Area Ration (FAR) as per load bearing capacity & available resources around the pockets.

8. CONCLUSIONS

In this paper there has been an attempt to study the possibilities of off-land structures on water. It discusses the need of floating structures and their scope of contribution towards development and growth of urban centers. A critical appraisal of the development and different design methods of off land floating structures has also been done. The off land structures or mega-floats have advantages over the traditional land reclamation solution for space creation in many ways like they will be cost effective in case of large water depth. It should be noted that the cost of imported sand for land reclamation in some countries has risen significantly and it may come a time that sand may not be even available from neighboring countries [14]. Floating structures may be environmental friendly as they will not damage the marine eco-system, or silt-up deep harbors or disrupt the tidal/ocean currents, they will be easy and fast to construct (components may be prefabricated at different shipyards and then brought to the site for assembling) and therefore sea-space can be speedily exploited. Psychologically the structures will be very efficient due to peaceful and clean environment.

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Experiences in Architecture Education Learning and Teaching Methodologies Topic: Teaching - Sharing or Enhancing the Learning

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ABSTRACT

Different lines of thought process and experimentation is the requirement of today's teaching. Black board teaching is one of the conventional systems of education. A challenge is to keep up the interest of the young learners, and making the learning more enjoyable process than just a submission. The short span of attention period of the pupil needs to be put up with work using rational thinking and hands on experiences. In order to succeed in the increasingly complex and rapidly growing techniques in the world, teachers need to equip themselves with effective thinking skills and update their knowledge to the present growing technology. Teaching strategies need mode of the process of critical thinking that would engage students in an active learning process i.e; the students are to be directly involved in the learning process through practical methods. This paper discusses the teaching and learning through different methods depending on the subject and also throws light on the effectiveness of the methods. Apart from the usual formal teacher-centered approaches, such as the fifty minute lecture, the active teaching and learning creates opportunities for interaction between teachers and students, discussions among students themselves, as well as between students and the subject materials. The visual and other methods of learning will create a strong bond to relate between the theoretical subjects taught to the practical design studios. This leads to promising opportunity for teacher to explore new methods and pedagogies for updating the learning and teaching strategies and also for further research to those educators who wish to pursue a deeper understanding of the process of learning to teach.

Keywords Conventional Method, New Method & Pedagogies, Practical Method- Active Learning, Active Teaching and Learning, Effective Teaching Skills

1. INTRODUCTION

Conventionally the lecture method is the most widely used instructional strategy for higher education; in colleges and universities. With the technology development and emergence of new teaching methodologies and strategies in the current day, the lecture still remains an important communication to students in different ways like: class room lecture, guest lecture, power point presentation, day long seminars, etc.

The conventional method had the usage of the black board. The composition of the board while lecturing was also to be in concern. The one hour of lecture session had to be compartmented in such a way that the whole hour would be divided for delivery of lecture, discussions ,question and answer session and ending up finally with the summary of the session taken during that particular hour.

In combination with active learning and teaching strategies, the conventional lecture is an effective method of communication. The advantages of the lecture approach is that it provides a way to communicate a large amount of information to many listeners (mass instruction)but the disadvantages being minimal feedback from students, assuming a minimal level of student understanding and often disengages the students (distraction) from the learning process causing information to be forgotten before the next lecture hour.

Faculty prefers the pressure for fewer lectures and to make the learning and teaching environments more interactive, to integrate technology into the learning experience and to use collaborative learning strategies when appropriate.

2. METHODOLOGY

Generally the lecture begins with general principles and eventually goes into detail with its applications. Instead, if the instruction begins with observations, book reviews, literature studies, site visits (on site instruction), case studies, etc. Solving of a live example, giving solutions , would be a preferable alternative in the teaching and learning process.

Teaching - Sharing or Enhancing the Learning

Apart from the usual formal teacher-centered approaches, such as the fifty minute/one hour lecture, the active teaching and learning creates opportunities for interaction between teachers and students, discussions among students themselves, as well as between students and the subject materials. The visual and other methods of learning will create a strong bond to relate between the theoretical subjects taught to the practical design studios.

These methods are learner centered; that they impose more responsibility on students for self-learning than the conventional lecture based approach. The students do more of research (research oriented projects) and their findings through the research are included into the existing information. They construct their own version of reality rather than just absorbing the versions presented by the faculty.

The following tips can make the lecture approach more effective

1. Focus on specific topic
2. Organize the points for clarity
3. Relation to exiting examples, illustrations/sketches to be shown.
4. Present more than one side of an issue and be sensitive to other perspectives
5. Be aware of the audience's reaction.
6. Enjoy ones presentation.

The methods would have more of student's involvement, discussions, questions and solutions (more than one). This would also lead to students working in groups (collaborative or cooperative learning).

The students are exposed to Case study based leaning, where they require to visit an existing project, document (take photographs, measurements ,sketches) the whole , observe, study the facts, critically analyze the advantages and disadvantages and finally incorporate it into their own design solution. This helps them to finally arrive on a design solution that is more functional and also aesthetical.

The case study approach works well as it acts as an extra support to the environmentally based design solution and it also stimulates critical thinking and awareness of multiple perspectives.

The exposure to site visits (factories, construction sites, etc) the students are practically aware of what's happening in the real world, more than just the theoretical approach. A hands-on workshop in the construction yard would create awareness for the students to real life applications.

Teaching method

Teaching Methods can be described as the way of achieving learning results. The selection of teaching methods is dependent on the students and the physiognomies of the learning condition. The selected teaching methods should support the completion of learning results.

Criteria's for an Appropriate Teaching method

Dynamic: Involve the students during learning hours i.e, they should be attentive and active by participating in the learning process. Listening to a lecture (passive learning) is often not the only and, perhaps not the best, way to attain the end product of learning, i.e. ; presentations, work well as a teaching method to get an outcomes of the type knowledge but for skill and competence learning your active portions for the learners should be incorporated in the teaching methods. Therefore, encouragement to the learners to participate in active learning rather than passive learning

Substantial: Teaching and Learning process are better achieved by linking the learning to a concrete application where knowledge, skills and competence are to be used. This way, learning is improved i.e.; the student is able to remember the information for a longer period and is able to apply the knowledge, skills and competence practically.

Support: support students in their learning process.

- Special lectures with demonstration.
- Preparation of check lists so that students can use during their active learning.
- Frequent one to one discussion with the students would help the students and it would lead to meaningful and enhanced learning. Discussions can happen as a group or personal depending on the exercise involved.

Social: a part of active learning. Engage the student's in more of social activities as part of the subject study. To learn with others is motivating and nurtures not only the learning process but also the students self-confidence and self-efficacy.

Technology and techniques should be used with care in the process. The use of technology should be in line with the instructional method which leads to the learning results.

3. DISCUSSION

Student's discussion

There are a various ways that stimulates a discussion: Faculty can commence on a chapter with a group discussion to refresh the students' memories about the subject matter. Faculty makes the students list critical points or emerging issues or they generate a set of questions focused on the specific subject. These strategies can also be used to help focus large and small group discussions.

Therefore a successful class discussion involves the faculty and students, where the faculty plans and also there is preparation on the part of the students. Faculty should communicate it very clearly to the students at the beginning of the session regarding the course expectations. As the faculty carefully plans the learning experience, the students also must comprehend to the subject taught.

Students usually find such activities energizing and are likely to engage more with the subject matter as a result. All students have previous experiences and knowledge of some kind and active strategies offer them the opportunity to make informal connections with things they have already learned. Active teaching and learning approaches will often yield unanticipated outcomes; there will be some learning that takes place, in other words, that has not been (and could not have been) planned for and this can be rewarding for both students and teachers.

Group work/discussions provide students with opportunities to learn from and support each other rather than mere formal, teacher-centered approaches.

By sharing knowledge and experiences, by encouraging students to think in different perspectives would make them react more critically.

Interactive teaching and learning sessions would encourage the students to become more self-directed and self-motivated.

4. RESULT

Benefits for students

Group projects/Group discussions can help students improve their skills that are increasingly important in the professional world. Properly structured group projects can reinforce skills that would enable the students to

- Breakdown the tasks into parts and steps
- Time management
- Refining the topic into simpler versions
- Development of communication skills.
- Face more complex problems.
- Share every individual perspective.
- Relate knowledge and skills.
- Risk management
- Develop new methods /approaches
- Develop one's own perspectives.
- oneself to get update their knowledge

This promotes meaningful teamwork and deep collaboration.

Benefits for faculty

Faculty can often assign complex, authentic problems to groups of students rather than individuals. Group work also introduces an unpredictable nature in teaching, since group work would solve problems in more diversified and interesting ways. Additionally, group assignments also would be appropriate so as to get more brain storming discussion and followed by diversified results in it.

5. CONCLUSIONS

The methods of instruction offer a promising opportunity for faculty to explore new methods, content, and pedagogies for teaching methodologies. The growing interest in faculty, the research about the methods, gives a deeper understanding of the process of learning to teach.

Faculty should first acquaint themselves with best practices such as providing extensive support and guidance when students are first introduced to the method and then further down withdraw the support as the students improve and get more experience and are confident of the usage of the new methods.

The goal should be to facilitate the students to become self- learners rather than them to depend on faculty as key source of information.

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Failure Process and Support Method of Roadways Excavated in Inclined Rockmass Strata

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A B S T R A C T

A three-dimensional geomechanical physical model is first constructed according to the similar simulation theory for a roadway excavated in inclined rock mass strata. The similar simulation model is then loaded in the laboratory using a self-designed YDM-E physical model testing device to observe the stress distribution and associated failure process of the roadway. For the roadway excavated in the rockmass strata with an inclined angle of 25°, it is found that cracks are initiated firstly in the rockmass located in the right-upper corner of the roadway and then propagate and coalesce resulting in that the rock located in the right-upper corner of the roadway detaches from the roof and falls into the roadway causing the collapse of the roadway. An unsymmetrical supporting scheme is then proposed to use cablebolts to supplement the regular support system of shotcrete lining and rockbolts for the roadway excavated in the inclined rock mass strata, which ensures the stability of the roadway while saving the supporting costs. In the proposed unsymmetrical supporting scheme, additional cablebolts are installed in the direction where the failures and rock falls easily occur during the tunnelling of the roadway in the inclined rockmass strata. The proposed unsymmetrical supporting scheme is finally tested in a roadway excavated in the inclined rockmass strata of a coal mine. According to the continuous monitoring results in three months, it is found that the convergence between the roof and the floor of the roadway and that between the lateral walls become stable in about two months after the tunnelling of the roadway, and the separation of rock mass strata in both the rockbolt-reinforced zone and the cablebolt-reinforced zone is small, which indicates that the proposed unsymmetrical support scheme can ensure the stability of the roadway excavated in the inclined rockmass strata while keeping the supporting costs economical.

Keywords Inclined Strata, Similar Simulation, Un-symmetrical Support, Roadway

1. INTRODUCTION

In long-wall coal mining method, roadways are usually excavated along the bedding coal strata. The stability of these roadways is essential for continuously mining the underground coal rescources. As the mining depth increases, the in-situ stress becomes bigger and bigger. To make sure the stability of the roadways in great depths, the supporting system must be designed carefully according to the raised in-situ stress and the excavation-induced stress concentrations. Many researchers have made great contributions to this field (Xu, 1990), which results in the development of various kinds of theories for the stability of the surrounding rockmass such as the natural balance arch theory, caving arch theory, elastic theory, elastic-plastic theory, EDZ (excavation disturbed zone) theory, and key strata theory. The theories for the support action of the installed rockbolt and other support system include the hanging theory, combined beam theory and compression reinforcement theory. These theories are derived under specific conditions in specific mines, and correspondingly their applications are restricted by their hypothetical conditions. Moreover, due to the complex and variable geological conditions in mining practices, the pure theoretical method can seldom provide a satisfactory solution.

Some researchers have conducted physical model tests, especially similar simulation experiment, to study the stability of the roadways excavated in the bedding rockmass strata. In the similar simulatio experiment, three-dimensional (3D) geomechanical models are built according to the similarity principles to study the stability of the tunnels in laboratories. Similar simulation experiment can visually imitate the stress concentration, deformation and failure of the surrounding rockmass induced by tunnel excavations and in-situ stresses. For example, He et al. (2010) studied the roadway excavation in the geologically horizontal strata at great depth using the similar simulation experiment. With the help of the infrared thermography and discrete Fourier transformation, they studied the response of the surrounding rock mass to the excavation process of the roadway, i.e. how the crack initiations and propagations and the pre-existing joints result in the collapse of the roadway during the excavation process. Meguid et al. (2008) reviewed the physical modelling including the similar simulation experiment of tunnelling in soft ground and summarized the advantages and disadvantages of each method available in literatures. Wu et al. (2009) built a similar simulation model using gypsum materials to study the mechanisms of rock burst during the excavation process of a roadway at great depths. According to their similar simulation experimental results, they developed a theoretical model for the zonal disintegration phenonmenon associated with rockburst based on the elasto-plastic analysis of the stress-strain behaviour of the surrounding rock mass using the improved Hoek-Brown strength criterion and the bilinear constitutive model. Liu et al. (2011) investigated the deformation of overlying coal seams above double-seam excavation using similar simulation experiments, which helped them draw the conclusion that due to the mining-induced stress redistribution, the pressure within overlying coal seams could be fully relieved to satisfy the statutory requirements of gas drainages.

Numerical methods have also been implemented to study the deformation and collapse process of the surrounding rockmass during the excavation of roadways under given geological conditions. The general-purpose finite element software ANSYS was used by Zhao et al. (2009) to study the rheological characteristics of a roadway excavated in a soft rockmass at great depth and the long-term mechanical behaviors of the support system of the roadway based on the generalized Kelvin creep model. Cai (2008) compared the simulated stress and yielding zone distributions in rockmasses around a circular tunnel using the explicit finite difference code FLAC and the implicit finite element code Phase 2. On the basis of his comparison, he stressed the influence of stress path on tunnel excavation response and made recommendation on how to choose appropriate tools and modeling strategies for tunnel excavation problems. The progressive failure processes of a circular opening excavated in a rock mass under biaxial compression and in a rock mass with pre-exisiting fractures are modelled by Wang et al. (2009) and Zheng et al. (2012), respectively, using the realistic failure process analysis code based on the finite element method. Recently, more and more researchers (Galli et al., 2004; Liu et al., 2008 and Lin et al., 2013) implemented full 3D finite element models to study the excavation process of both shallow and deep tunnels including the step-by-step excavation and support installation processes, the interaction between the surrounding rock mass and the support system, i.e. concrete/shotcrete lining and rockbolts, and the elasto-plastic behaviour of the surrounding rockmass and the installed support.

Each method reviewed above can reflect, to some extent, the stress redistribution and concentration, the deformation, and the crack initiation, propagation and coalescence in the surrounding rock mass during the excavation process of the roadway. In this paper, a 3D geomechanical physical model is firstly constructed in laboratory using the similar simulation method for the excavation of a roadway in inclined rockmass strata. The 3D geomechanical physical model is then loaded from the top bounday to visually reflect the behaviour of the roadway excavated in the inclined rockmass strata. After that, a support scheme is proposed for the roadway excavated in the inclined rockmass strata on the basis of the experiment results. Finally, the proposed support scheme for the roadway excavated in inclined rockmass is tested in the field to ensure the stability of the roadway under the real geological conditions.

2. INCLINED ROCKMASS STRATA AROUND THE ROADWAY

The prototype of the similar simulation experimemt is a roadway excavated in the No. 8th mine of the Hebi Company of the Henan Coal Chemical Industry Group Co. Ltd, Henan Province, China. The roadway is excavated in the rockmass strata in the central region of the 3rd mining level of the mine, which is 530 m below the sea level. The roadway connects the dummy inclined shaft of ventilation at the southern end with the dummy inclined shaft of cable transportion at the northern end. The length of the roadway is 450m and its design life is 30 years. The roadway is designed for coal transportion, ventilation, and mining workers' access to the No. 3003 working face.

Rockmass stratum name	Thickness (m)	Legend
Sandy mudstone	6.40	
Fine sandstone	3.80	
Siltstone	3.90	
Mudstone	0.95	
Limestone	6.60	
Sandy mudstone	6.25	
Coal	0.45	
Mudstone	2.30	
Fine sandstone	2.10	
Mudstone	1.90	
Fine mudstone	3.26	

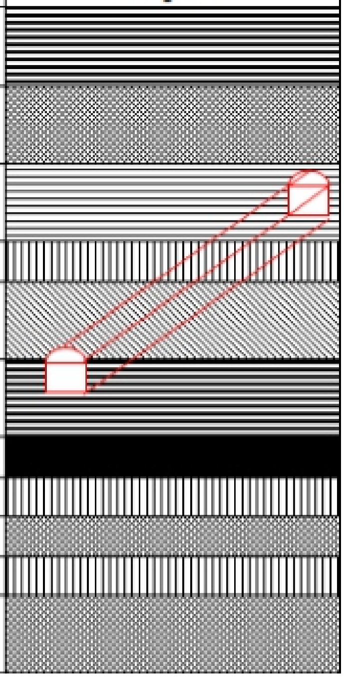


Figure 1. Columnar section of the rockmass strata surrounding the roadway to be excavated

The mine where the roadway is excavated has a simple monoclinal structure and the inclined angle of the rockmass strata is 25°. The roadway has a cover depth of 640 m and an inverse horse shoe shape with vertical lateral walls and semi-circle arch. The height of the vertical walls of the roadway is 1.3 m, the width is 3.8 m, and the total height is 3.2 m. The roadway is first excavated in the sandy mudstone with a thickness of 6.25 m in average. As shown in Fig. 1, the immediate roof of the roadway is a stratum of hard limestone with a thickness of 6.6 m in average. The immediate floor is mainly a mudstone with a thin coal stratum. After that, the roadway passes through the limestone and mudstone, and is mainly excavated in the hard siltstone, as shown in Fig. 1. In this case, the roof and lateral sidewalls are fine sandstone and hard siltstone, respectively. The thicknesses of the fine sandstone and the hard siltstone are 3.8 m and 3.9 m, respectively. The immediate floor is the soft mudstone with a thickness of 0.95 m.

3. SIMILAR MATERIAL SIMULATION EXPERIMENT

According to the similar theory (Li, 1987), the materials adopted in the similar simulation experiment must have the physical-mechanical properties similar to those of the prototypes, which requires similarities hold true for the geometries, densities, strengthes, stresses, forces, time, etc, of the materials in the prototypes and those used in the similar simulation experiment.

3.1. Selection of Similar Ratio

The similar ratio should be chosen on the basis of the size of the model frame, the size of the prototype to be modelled, and the capacity of the loading system. Generally, the possible biggest similar ratio should be selected so that the protoype can be modelled as accurately as possible and the experimental results can realistically reflect what happen in the prototype. Following these considerations, the similar ratio of the sizes of the similarity model and the prototype is chosen as 1:20, and that of the bulk density is 1:1.5. Correspondingly, the similar ratios of the strength / stress, load, and Poisson's ratio can be calculated, which are 1:30, 1:12000, and 1:1, respectively. The size of the similar simulation model is 1600 mm (length) × 1600 mm (width) × 400 mm (thickness), which corresponds to 32 m (length) × 32 m (width) × 8 m (thickness) of the rockmass in field.

3.2. Selection of Similar Material

The similar material is made of water, cement, gypsum, sand and borax. Among them, the sand is the main component, the cement and the gypsum are used as bonding materials, and the borax is working as retarder. According to the physical-mechanical properties of the rock mass surrounding the roadway, the quantity of each component is calculated, which is summarized into Table 1. These components are then mixed with each other to model the rock mass strata in the field. In Table 1, the mixture ratio stands for that of the weights of sand : cement : gypsum. For each rock mass stratum, the total quantity of mixture used in the similar simulation experiment is first calculated according to the thickness and density of the rock mass strata, and the width and similar ratio of the similar simulation model. The quanities of the sand, cement and gypsum are then calculated according the mixture ratio. For example, for the sandy mudstone, the mixture ratio is 90:5:5, which means the quantities of the sand, cement and gypsum are 0.9, 0.05 and 0.05, respectively, times the total quantity. The quantity of the water is 0.1 times the total quantity while that of the borax is 0.01 times the quantify of the water.

Table 1. Mixture ratio and material quantity for the similar material simulation experiment

Strata	No. of ratio	Sand/kg	cement/kg	gypsum/kg	water/kg	borax/kg
Sandy mudstone	90:05:05	221.36	12.3	12.3	24.6	0.25
Fine sandstone	80:05:05	170.38	21.3	21.3	21.3	0.21
Siltstone	90:03:07	217.39	7.25	16.91	24.15	0.24
Mudstone	90:05:05	42.34	2.35	2.35	4.7	0.05
Limestone	70:05:05	294.5	63.11	63.11	42.07	0.42
Sandy mudstone	90:05:05	291.13	16.17	16.17	32.35	0.32
Mudstone	90:05:05	99.46	5.53	5.53	11.05	0.11
Fine sandstone	80:05:05	62.23	7.78	7.78	7.78	0.08
Mudstone	90:05:05	53.73	2.98	2.98	5.97	0.06
Fine sandstone	80:05:05	68.64	8.58	8.58	8.58	0.09

3.3. Similar Simulation Experiment Setup

The similar simulation experiment is conducted using the YDM-E physical model testing device developed jointly by Henan Polytechnic University and the Third Institute of Engineering, Army of General Staff of PLA, which is shown in Fig. 2. The physical model testing device consists of five components, namely, model frame, mould for testing samples, transportation vehicle, hydraulic control system and data acquisition system. The induced stresses are logged using the YJZ-16A intelligence digital stress instrument developed by Beijing Tairui Jinxing Instrument Ltd. During the model preparation, strain gauges are buried into the model, as shown in Fig. 3, which are then calibrated to determine the corresponding stress to each unit of strain. After each loading, it is required to wait 30 min before the measurement is conducted so that the stress transfer process is completed at the time of the measurement. The data from each measurement point are converted by the YJZ-16A intelligent digital strain instrument into digital values, which can be directly read from computers. The obtained data are finally converted into the actual stresses in the rockmass according to the stress and strain relation calibrated previously and the similarity ratio.

Figure 2. Similar simulation experimental setup

Since the rockmass strata is inclined to the horizontal direction at an angle of 25°, the model frame is rotated 25° for the convenience during the model preparation. The similar materials prepared according to Table 1 are then filled into the model frame layer by layer and densified using a metal block. Each layer is separated by wood powder or mica powder from the adjacent layers. If the thickness of a layer is relatively big, the layer is filled through a few thin sub-layers. Finally, the model frame is brought back to its normal position. The prepared final similar simulation model is shown in Fig. 2. As can be seen from Fig. 2, there are two similar simulation models installed in the model frame: the roadway in the left model is supported while that in the right model is unsupported. The support system of the roadway in the left model includes the shotcrete lining and rockbolts depicted in Fig. 7, which are symmetrical to the central line of the roadway, but does not include the unsymmetrical cablebolts in Fig. 7. The prepared models are left in the model frame to dry for 15 days before the similar simulation experiment should begin.

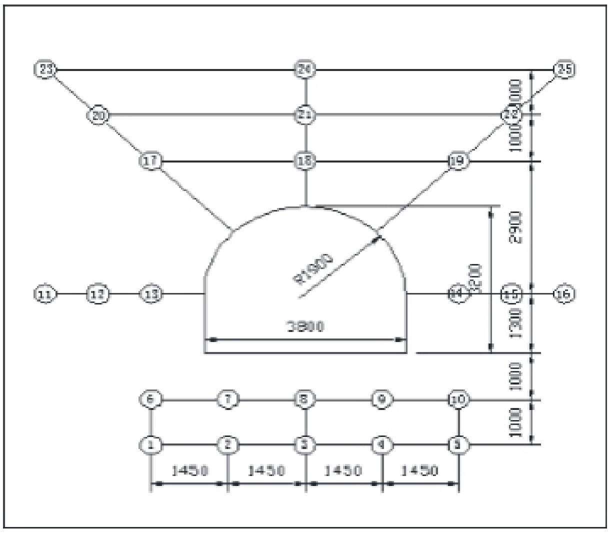
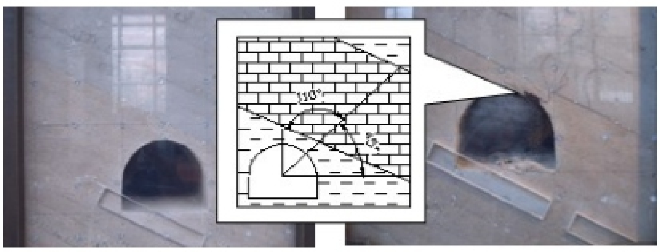


Figure 3. Distribution of monitoring points in the similar simulation model

4. EXPERIMENTAL RESULTS

During the similar simulation experiment, vertical loads are applied on the top boundary of the model through the hydraulic control system of the YDM-E physical model testing apparatus and the lateral sides are restricted to move in the horizontal direction, which applies a reaction force (i.e. passive loading) in the lateral sides of the model. Fig. 4 records the deformation and failure processes of the roadways when the model is subjected to various loads. Since the characteristics of the deformation and failure processes of the roadway supported using symmetrical shotcrete lining and rock bolt are similar to those of the unsupported roadway. Only the deformation and failure processes of the unsupported roadway are described here in detail. It can be seen from Fig. 4 that, as the load increases to1 6.82 MPa, cracks are firstly initiated in the rock mass located in the right-upper corner of the roadway, i.e. about 45° from the horizontal direction (Fig. 4 b). More and more cracks nucleate there as the load increases (Fig. 4 c). When the load is increased to 11.34 MPa, the rock located in the right-upper corner of the roadway detaches from the roof and falls into the roadway (Fig. 4 d). With the load further increasing, more and more rock falls from there (Fig. 4 e). Finally, when the load increases to 15.86 MPa, the roadway is destroyed completely, as shown in Fig. 4 f.



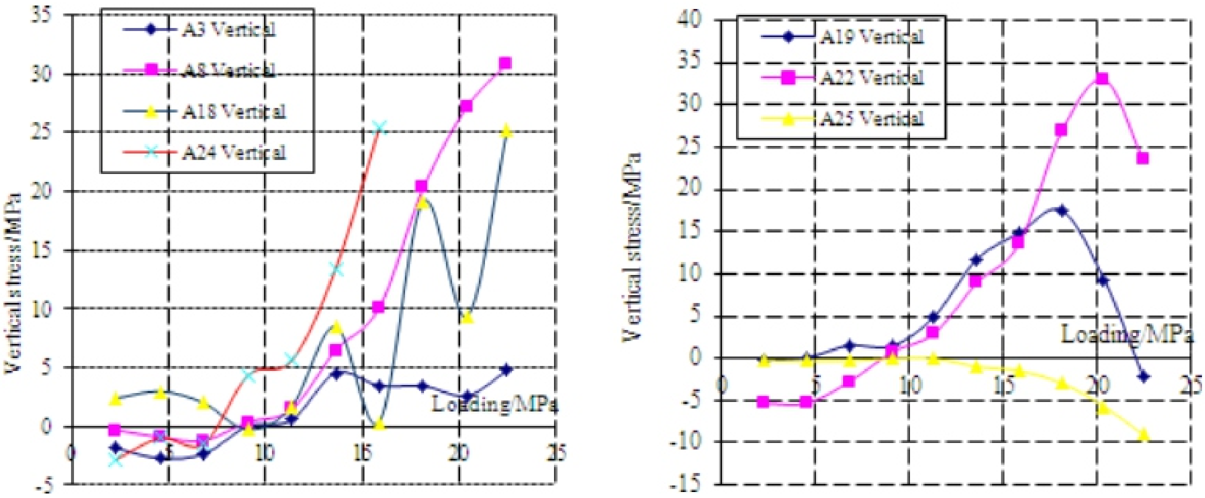
a) 2.3 MPa b) 6.82 MPa

c) 9.08 MP a d) 11.34 MPa

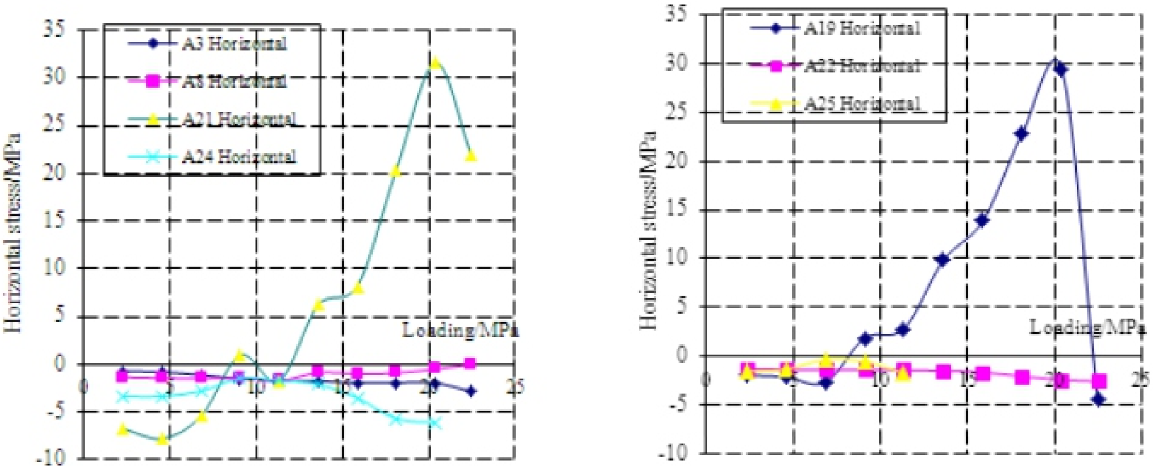
e) 13.6 MP a f) 15.86 MPa

Figure 4. Failure process of the roadway as the applied load increases

The stress distribution in the rock mass surrounding the roadway is the main factor controlling the deformation and failure process of the roadway. As introduced above, the strain gauges are buried into the similar simulation model to log the stresses at the measurement points depicted in Fig. 3 using the YJZ-16A intelligence digital stress instrument, which can be used to investigate evolutional laws of the stresses at the measurement points as the applied load increases. Fig. 5 records the variation of the vertical and horizontal stresses monitored at the key measuring points along the central line of the roadway and the 45° inclined line. In Fig. 5, the compressive stress is regarded as negative and the tensile stress is regarded as positive. As shown in Fig. 3, the measuring point A3 is located at 2m from the floor of the roadway along the central line of the roadway. The vertical stress at A3 is compressive at the beginning of the loading (Fig. 5 a). As the load increases to 10MPa, the vertical stress at A3 changes to become tensile from the original compressive stress (Fig. 5 a) although the horizontal stress at A3 has not changed obviously (Fig. 5 c) during the loading process since it is 2m away from the roadway opening (Fig. 3). Thus, the floor of the roadway has the tendency of upheave. The monitoring point A8 is also located at the floor of the roadway but it is located at 1 m above A3 along the central line. It can be seen from Fig. 5 a) and c), the variation of the vertical and horizontal stresses at A8 is similar to those at A3 at the beginning of the loading. However, as the load increases to 10MPa, the tensile stress at A8 increases rapidly and correspondingly the floor of the roadway heaves up, which can also be observed in Fig. 4. The monitoring points A18, A21 and A24 is located at the roof of the roadway along the central line. It can be seen from Fig. 5 a) and c) that the vertical stresses at A18 and A24 and the horizontal stresses at A21 vary rapidly at the later stage of the loading, especially at the measuring point A18, the vertical stress repeatly reverses. The reason is probably that A18 is located at the roof of the roadway closest to the opening. Moreover, the falling of rock into the opening cause the separation of layers around the measuring point A18, which makes the data there become invalid and the tensile stresses there continuously be released and accumulated. That is why the vertical stress at A18 in Fig. 5 a) shows the obvious oscillation during the collapse process of the roadway. The measuring points A19, A22, and A25 are located in the right-upper roof of the roadway, i.e. 45° from the horizontal direction. It can be seen from Fig. 5 b) and d), at the later stage of the loading, there are big tensile stresses in both horizontal and vertical directions at the monitoring point A19, which indicates that the damage and failure easily occur there. This is consistent with the rock falls photographed in Fig. 4. The falling of rock into the opening causes the separation of layers around the measuring point A19, which makes the measuring point A19 becomes invalid after the rock falls. That is why big changes of horizontal and vertical stresses at the measuring point A19 are recorded after the rock falls. In comparison, both vertical and horizontal stresses monitored at the measuring point A25 remain compressive since A25 is much further away from the opening of the roadway than A19. The measuring point A22 is located between A19 and A25. Correspondingly, the stresses at A22 are between those at A19 and A25.

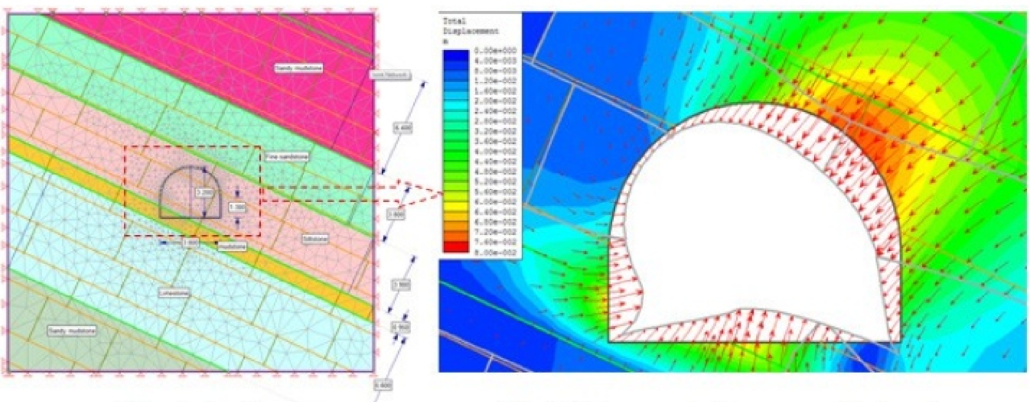


a) Variation of vertical stresses at the monitoring points along the central line
b) Variation of vertical stresses at the monitoring points along the 45° line



c) Variation of horizontal stresses at the monitoring points along the central line
d) Variation of horizontal stresses at the monitoring points along the 45° line

Figure 5. Variation of the vertical and horizontal stresses in the rockmass surrounding the roadway as the applied load increases (the labels of the monitoring points correspond to those depicted in Fig. 3)



a) Numerical model b) Total displacements of rock mass surrounding the roadway

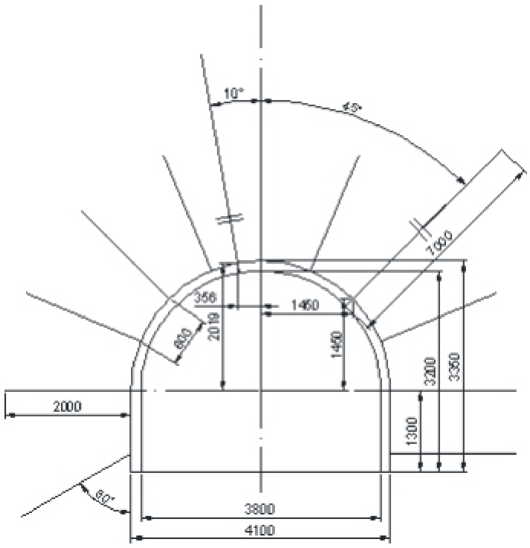
Figure 6. Numerical modelling of the deformation process of the roadway as the applied load increases to 11.34 MPa, which corresponds to the loads in Fig. 4 d)

Therefore, it can be seen from the experimental and numerical results presented above that due to the inclined orientation of the rockmass strata, the roadway is loaded unsymmetrically, which results in that the cracks are first nucleated and initiated at the right-upper corner of the roadway and then propagate and coalesce there resulting in rock falls there and the unsymmetrical failure pattern of the roadway. Moreover, it is found from the similar simulation experiment that even if the roadway is supported using symmetrical shotcrete lining and rock bolts, there are few differences in the observed deformation and failure patterns of the roadway. To ensure the stability of the roadway, an unsymmetrical support scheme may be designed for the roadway to save the supporting costs. The unsymmetrical support scheme can be implemented using cablebolts since each cablebolt can only reinforce the rockmass surrounding the installing location. The direction of the installed cablebolts should be consistent with the direction where the surrounding rock mass may easily fail due to the unsymmetrical loads caused by the inclined rockmass strata, i.e. the about 45 degree from the horizontal direction for this study. However, the supporting direction depends on the factors such as the inclined angle of the rockmass strata, physical-mechanical properties of the rockmasses, and in-situ stresses, where further studies are needed.

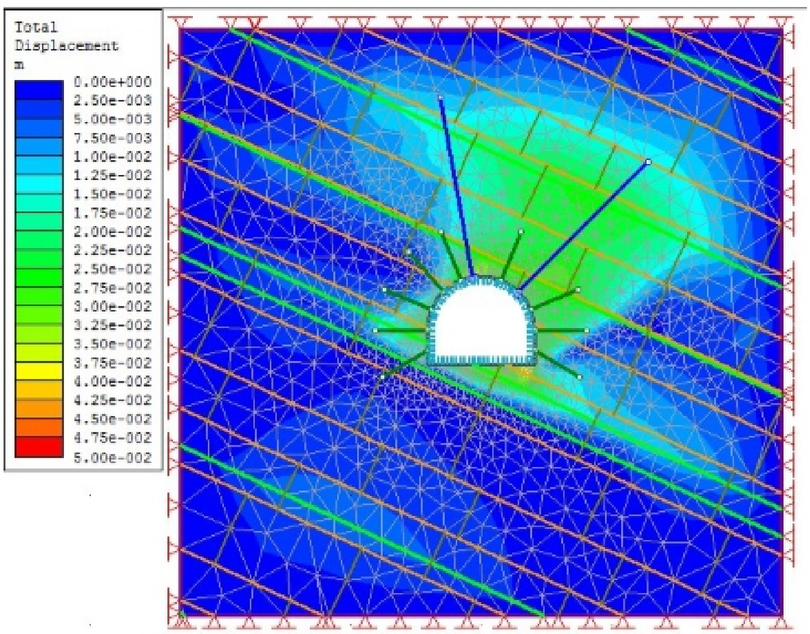
In order to validate the similar simulation experimental results, a simple finite element model is constructed using Rocscience Phase2D, as shown in Fig. 6 a. The roadway is then excavated in the rock wass with various in-situ stresses to observe the deformaton process of the roadway. Fig. 6 depicts the modelled total displacement of the rock mass surrounding the roadway excavation. It can be seen from Fig. 6 b that, when the in-situ stress increases to 11.34 MPa, which corresponds to the loads in Fig. 4 d, the rock mass located in the right-upper corner of the roadway moves greatest towards the roadway excavation and the floor heaves up. It is obvious that the characteristics of the modelled deformation process of the roadway support those of the deformation and failure processes of the roadway observed in the similar simulation experiment although more sophisticated numerical analyses may be needed.

5. DESIGN OF UNSYMMETRICAL SUPPORT SCHEME

According to the results from the similar simulation experiments introduced above, an unsymmetrical support scheme is adopted to support the roadway excavated in the inclined rockmass strata. Rockbolts and cablebolts are used as the main reinforing elements. The selected rockbolts are screw thread steel anchor with a diameter of 20 mmmm and a length of 2000 mm, which are installed symmetrically along the opening of the roadway with a spacing grid of 800 mm × 800 mm, as shown in Fig. 7 a. The rockbolts are fixed using 2 volumes of CK2335 resin grouting agents. As shown in Fig. 7 a, the layout of the cablebolts is unsymmetrical: one is installed at the direction of 45° from the horizontal direction, which is the main direction where the cracks are first nucleated and initiated, and propagate and coalesce resulting in the rock falls. The other cablebolt is installed at the direction of 10° away from the central line of the roadway opening, which is the location where the rock falls finally propagate to. The chosen cablebolts are the low relaxation steel strand (1 × 7mmmm) with a diameter of 17.18 mmmm and a length of 7000 mm. The cablebolts are fixed using 1 volume of CK2335 and 3 volumes of Z2335 resin grouting agents. In sum, the rockbolts are installed symmetrical to the central line of the roadway opening while the cablebolts are mainly installed in the regions where the major rockfalls are expected to occur according to the similar simulation experiments, as shown in Fig. 7 a. Along the axis of the roadway, cablebolts are installed in every two rows of rockbolts and thus the spacing of cablebolts is 1600 mm since the rockbolts are distributed in a spacing grid of 800 mm × 800 mm.



a) Unsymmetrical support system



a) Numerical validation of the unsymmetrical support system

Figure 7. Unsymmetrical support design for a roadway excavated in inclined rock mass strata

To check whether the unsymmetrical support scheme can satisfy the requirements, a simple finite element analysis is conducted again using Rocscience Phase2D to analyze the deformation behaviour of the roadway supported using the unsymmetrical support system, as shown in Fig. 7 b. Through comparing the total displacement fields depicted in Fig. 6 b and Fig. 7 b, it is obvious that the unsymmetrical support system successfuly limits the deformation and then failure of the rock mass in the right-upper corner of the roadway excavated in the inclined rockmass strata.

6. FIELD EXPERIMENTS

Field experiments are conducted in the No. 3003 roadway excavated in the inclined rockmass strata in the No. 8th Coal Mine of Hebi Mining Corporation, Henan Province, China. The roadway is located on the top of limestone and excavated in mudstone and siltstone. As shown in Fig. 1, the mudstone has a thickness of 0.95 m, which is located at the floor of the excavated roadway. The roof and lateral walls of the roadway are harder siltstone with an average thickness of 3.9 m and fine sandstone with an average thickness of 3.8m, respectively. The cover depth of the roadway is 640m from the ground surface. In light of the similar simulation experimental results, an unsymmetrical support scheme has been designed in Section 5 for the No. 3003 roadway. During the field experiments, the convergence of the roadway between the roof and floor and that between the lateral walls are monitored. Moreover, a few measuring points are installed in the areas reinforced by rockbolts and cablebolts to monitor the displacements there, too.6.1. Convergence between the roof and floor of the roadway and that between the lateral walls.

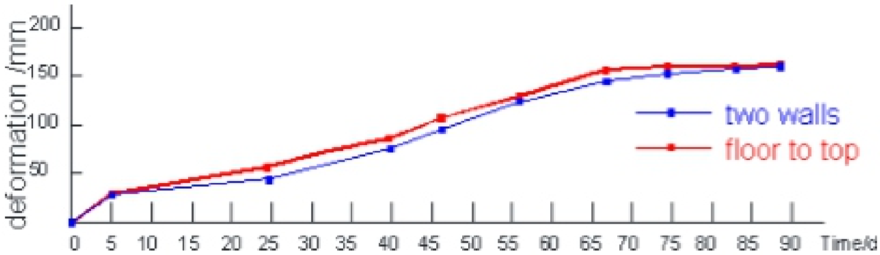


Figure 8. Convergence between the roof and floor and that between the lateral walls of the roadway as the time increases

During the tunnelling of the roadway, the displacements at the roof, floor and lateral walls of the roadway are monitored periodically to record the convergence of the roadway opening. After continuously monitoring for about 3 months, the variational law of the convergence of the roadway opening is basically revealed. Fig. 8 depicts the convergence between the roof and floor and that between the lateral walls of the roadway according to the monitored results at a few measuring points. It can be seen that during the first 5 days, i.e. immediately after the tunnelling of the roadway, the opening of the roadway convergences rapidly and the observed deformation is signficant. The deformation rate slows down after 5 days and the deformation gradually increases from 5 days till about 65 days. The convergence becomes basically stable in about two months after the tunnelling of the roadway. In the third month during monitoring, almost no convergence is observed between the floor and roof of the roadway opening and little convergence is observed between the lateral walls of the roadway opening. On the 90th day of the monitoring, the recorded maximum convergence between the roof and floor and that between the lateral walls are 160.9 mm and 170.43 mm, respectively. The rate of the convergence between the roof and floor of the roadway and that between the lateral walls are 0.08 mm per day and 0.09 mm per day, respectively, in average in the last month of the monitoring, which reveals that the roadway is now stable and the designed unsymmetrical support scheme satistifies the supporting requirements of the roadway.

6.2. Roof Separations Monitored Along the Axis of the Roadway

During the field experiments, two types of additional measuring points are installed in the surrounding rockmass 1.9 m and 6.9 m from the excavation profile of the roadway along the axis of the roadway, which are named as shallow and deep measuring points, respectively. Since the lengths of the installed rockbolts and cablebolts are 2000 mm and 7000 mm, respectively, the installed shallow and deep measuring points are actually used to monitor the displacement and separation of the rockmass strata in the rockbolt-reinforced and cablebolt-reinforced areas, respectively. Since the monitored rock mass stratum separation rate is very small, only the final values of the separations of the rockmass strata at the shallow and deep monitoring points are listed in Table 2. As can be seen from Table 2, the maximum separation of the rockmass strata inside the rockbolt-reinforced zone along the axis of the roadway is 5mm, the minimum separation is 2 mm, and the average separation is 3.75 mm. In the cablebolt-reinforced zone, the maximum separation is 12mm, the minimum separation is 7mm, and the average separation is 9.75mm. Thus, it is obvious that the separation of the rockmass strata in the rockbolt-reinforced zone is smaller than that in the cablebolt-reinforced zone. Moreover, it is obvious that all of the monitored separations are small, which satisfies the requirements of the roof separations specified in the mining legislations of the mine. Correspondingly, it is concluded that the roadway is stable, which reveals that the proposed unsymmetrical support scheme can ensure the stability of the roadway excavated in the inclined rockmass strata.

Table 2. Roof separations monitored at various locations along the axis of the roadway

Testing point number	No. 1		No. 2		No. 3		No. 4	
	Shallow	Deep	Shallow	Deep	Shallow	Deep	Shallow	Deep
Initial/mm	0	0	0	0	0	0	0	0
Final/mm	3	7	5	11	2	9	5	12
Abscission/mm	3	7	5	11	2	9	5	12

CONCLUSIONS

In this study, 3D similar material simulation experiments are first conducted to study the stress distribution and failure process of a roadway excavated in the inclined rockmass strata. For the roadway excavated in the rockmass strata with an inclinded angle of 25°, it is found that cracks are initiated firstly in the rockmass located in the right-upper roof of the roadway, i.e. about 45° from the horizontal direction. As the applied vertical load increases, more and more cracks nucleate, propagate and coalesce there. The rock located in the right-upper roof of the roadway then detaches from the roof and falls into the roadway resulting in the collapse of the roadway. On the basis of the similar material simulation experimental results, an unsymmetrical supporting scheme is then proposed to use rockbolts and cablebolts to support the roadway excavated in the inclined rockmass strata ensuring the instability of the roadway while keeping the supporting costs economical. In the proposed unsymmetrical supporting scheme, additional cablebolts are installed in the direction where the failures and rock falls easily occur during the tunnelling of the roadway in the inclined rockmass strata. The proposed unsymmetrical supporting scheme is finally tested in a roadway excavated in the inclined rockmass strata in a coal mine. During the field experiments, several measuring points are distributed in the excavation profile of the roadway and buried in the surrounding rock mass to monitor the convergence of the roadway and the rockmass strata separation in the rockbolt-reinforced zone and the cablebolt-reinforce zone, respectively. According to the continuously monitoring results in three months, it is found that the convergence between the roof and floor of the roadway and that between the lateral walls become stable in about two months after the tunnelling of the roadway, and the separation of rock mass strata in both the rockbolt-reinforced zone and the cablebolt-reinforced zone is small, which indicates that the proposed unsymmetrical support scheme can ensure the stability of the roadway excavated in the inclined rockmass strata while keeping the supporting costs economical although further studies may be needed for the stability of the roadway during vehicle movement.

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Effectiveness Evaluation of Zali bet as Reinforcement in Concrete Beam

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A B S T R A C T

In this paper the yield strength, ultimate strength and modulus of elasticity of a rattan (Calamus guruba, local name in Bangladesh is "Zali bet") were measured. After that the performance of some concrete beams was justified where this rattan was used both as flexural and shear reinforcements. Total eleven beams were tested in this study. Among them seven were singly reinforced beams and were used in flexure test. And the other four beams were prepared for shear test. Concrete properties were kept constant leaving reinforcement the only variable. The lengths were 500 mm for flexure test and 1025 mm for shear test having the same width and height. The test results were compared with non-reinforced concrete beams. The results were analyzed using the conventional moment equations and bond stress. Relations between the experimental and theoretical values were made. Finally design of rattan-reinforced beam was also made to understand its contribution physically.

Keywords Bond-stress, Deflection, Flexure, Moment Equation, Shear, Zali

1. INTRODUCTION

Rattans are extensively used for manufacturing furniture and baskets, it accepts paints and stains like wood, and it is available in many colors; it can be worked into many styles. It is a non corrosive material in nature and thermally non-conductive. Rattan is flexible, and is very easy to shape and form. Due to strength and durability it has been used as a construction material in certain areas for centuries, but its application as reinforcement in concrete had received little attention. There are certain limitations to the use of rattan cane; one of them is the starchy interior is attractive to insects. For long lasting structures, it is important to treat these materials against rots and insects. Most importantly, it is important to dry before it is used in construction, rattan cane furniture have been found to have a life span of over 50 years and this will depend on the exposure conditions. In order to design any structural component efficiently, it is jenkinsiana). Chowdhury [2] studied on the physical and mechanical properties of this rattan. Based on that study another research was conducted by Mahzuz et al [6] focusing on its bond strength with concrete. In that study an experiment was also made to quantify the effectiveness of this rattan as reinforcement in concrete beam. But taking "Zali bet" (Calamus guruba) no study was seen to be conducted. Generally it is very widely used in handicraft sector in Bangladesh which contains a great heritage value. Due to its durability and strength in products it is an encouraging subject for research. Moreover no study was found considering it as reinforcement in concrete beam. Keeping this in mind the bond strength of it with concrete was also tested by Pull-out test. Since Zali bet has smooth surface therefore its tensile strength may not be fully utilized. Therefore three types of specimen of Zali bet were prepared, a) Normal rattan, b) Rattan having ring of GI wire (25 mm pitch) and c) Rattan having spiral of GI wire (25 mm pitch).

The corresponding result was 0.22, 0.46, 0.28 MPa respectively. This means Rattan having ring of GI wire (25 mm pitch) showed the best performance [7]. Using the information got from that research paper an attempt is made in this study to utilize "Zali bet" as reinforcement in a load bearing member like beam. As a preliminary work it is expected that this will provide some initial data for future researchers interested in this field.

2. TESTING PROCEDURE

The research was performed using the concrete where Ordinary Portland Cement (ASTM Type-1) was used at a mixing ratio of 1:2:3 having the same water-cement (w/c) ratio (0.5). The fineness modulus (FM) of sand was 2.8 (Table-1). At least three year old rattan-Zali bet (Calamus guruba) was used at the study. The rattan samples were straight and free from any kind of physical fault, such as twist, insect attack, deformation etc.

2.1. Tension test of Zali:

In this study Zali samples were tested to find out the yield strength, ultimate strength and modulus of elasticity. Three years old five fault-free straight samples were selected for tension test.

2.2. Flexure tests

All the flexure tests were conducted using the same beam size of 500 mm length (distance between supports was 450 mm), 150 mm width and 150 mm depth (Figure 1). Clear cover was 12.5 mm and effective depth was 125 mm. Cross-sectional area of each rattan piece was measured at three different locations (two sides and middle), and then average area was calculated.

All the beams were singly reinforced, that is no reinforcement at top was used. Each beam was reinforced by three 10 mm bars as shear reinforcement (Fy = 414 MPa) at a center to center distance 150 mm. The shear strength of each beam was kept constant at each beam leaving the flexural strength as variable. Due to smooth surface of rattan it might offer low bond strength with concrete. To increase the bond strength Galvanized Iron (GI) wire (1.75 mm diameter) was used in ring form with rattan. The spacing of ring of GI was 25 mm. At the end of ring the GI wire was twisted and extended 12.5 mm. Such formation of GI wire with rattan increased the bond strength with concrete. The reason behind this phenomenon was the twisted and extended portion of GI wire that produced rough surface. And this rough portion offered extra resistance against the pulling force. In such case the bond stress was 0.46 MPa, bond stress without GI wire was 0.22 MPa [6].

Table 1. Properties of the concrete used

Cement Type	FM of Sand	Water Cement Ratio	Unit weight of stone (Kg/m^3)	Mix ratio	28 days Compressive	Unit weight of Concrete (Kg/m^3)
OPC- ASTM Type-1	2.8	0.5	1500±100 (19 mm downgraded)	01:02:03	24.5±3	2357±200

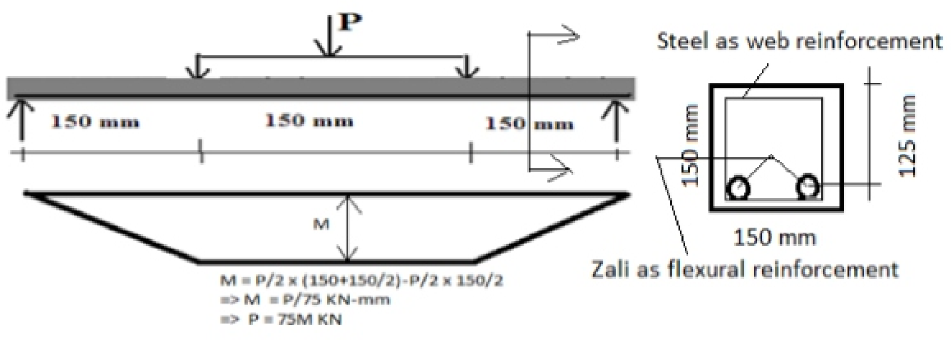

Figure 1. Flexure Test the setup, beam cross-section and bending moment diagram (neglecting the self weight) for beam

A total number of seven beams were prepared out of which:

a) six beams of were prepared with varying percentage of reinforcement (ρ) using rattan (Zali bet) as flexural reinforcement as shown in Figure 2,

b) lastly one beam is prepared using no flexural reinforcement.

Curing was done in a water reservoir for 28 days and each beam was tested using two point loading technique at Universal Testing Machine (UTM). To get the information of deformation, a strain gauge was used at the mid point of the beams during the test of beams.

Figure 2. Flexural reinforcement of Beam

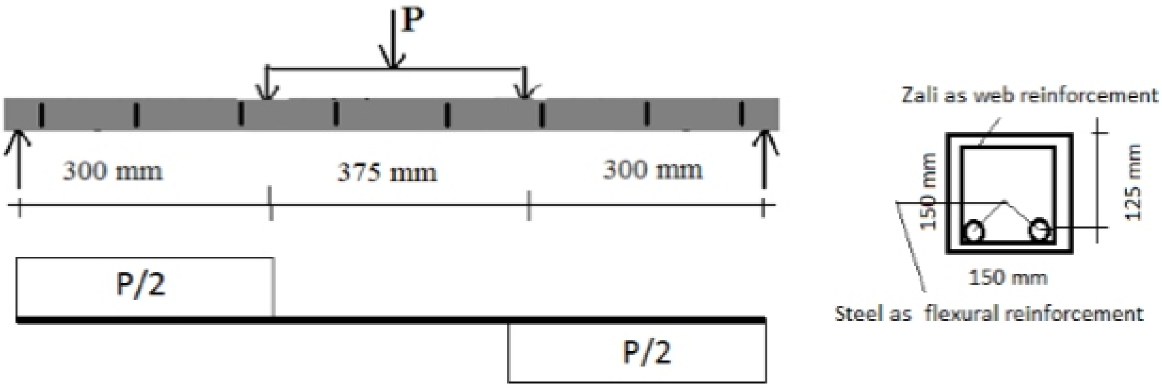

Figure 3. Setup for beam for Shear Test, beam cross section and shear force diagram (neglecting the self weight) for beam

2.3. Shear Test of Concrete Beam

All the shear tests were conducted using a beam size of 1025 mm length, 150 mm width and 150 mm depth (Figure 3). Clear cover was 12.5 mm and effective depth was 125 mm.

Four beams were prepared for this test, out of which:

a) three beams were prepared where rattan (Zali) was used as shear reinforcement,

b) the rest one was prepared without any shear reinforcement for comparison.

Since this test is related to shear, therefore the flexural capacity of the beams were kept stronger that that of shear capacity. To confirm this situation two 10 mm dia and two 16 mm dia rebar (having fy = 500 MPa) were used at top and bottom respectively as flexural reinforcement. All the rattan samples were cut at a length of 125 mm. The spacing of rings rattan were different (75mm, 100 mm and 125 mm), but the cross-sectional area was same. The Zali as Shear Reinforcement in concrete beam is shown at Figure 4. The beams were tested at UTM as two points loading system. The specification of the Concrete used was as described at the Sub-heading 2.0.

Figure 4. Zali as Shear Reinforcement in concrete beam (full view)

3. EXPERIMENTAL RESULT AND DISCUSSION

3.1. Results on Tension test of Zali

Offset method was used to find the yield strength of rattan. Generally the offset value for yield strength of cast iron, wood (parallel to grain) and concrete is 0.02%-0.05%, 0.05% and 0.01%-0.02% respectively [1]. Since no specific offset value was found for rattan therefore the value was taken 0.05%. The yield strength of Zali was found 52 MPa. The stress-strain curves are shown from Figure 5-9. The ultimate strength and modulus of elasticity was 71 MPa and 8805 MPa respectively (Table-2).

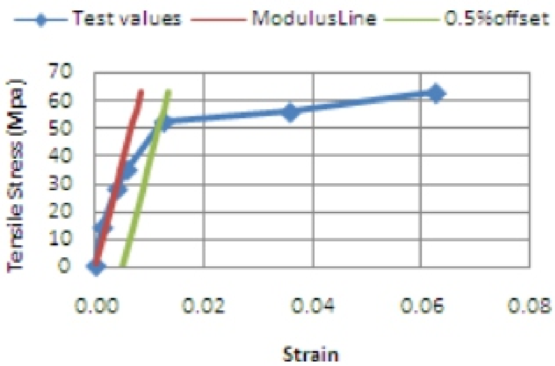

Figure 5. Stress-strain diagram of sample-1

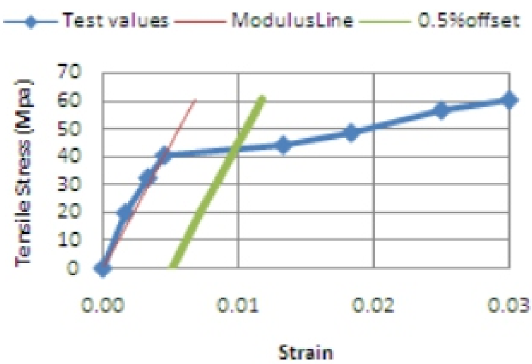

Figure 6. Stress-strain diagram of sample-2

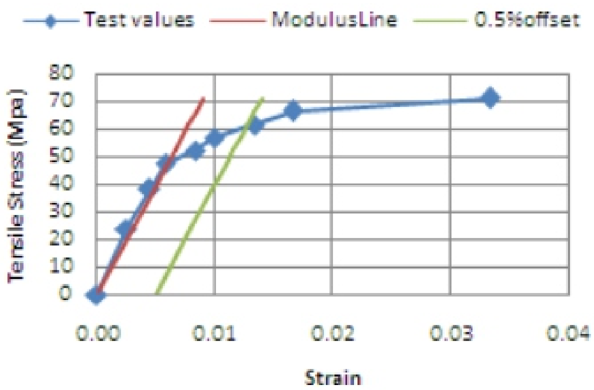

Figure 7. Stress-strain diagram of sample-3

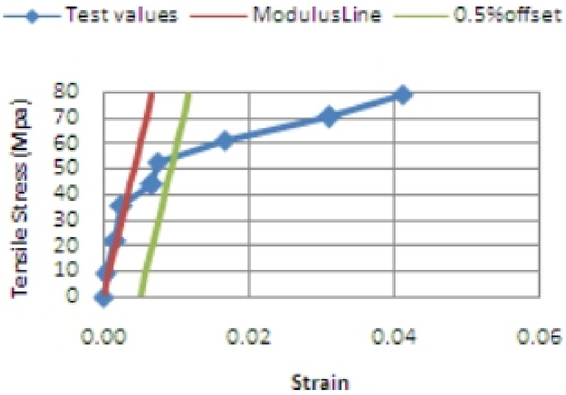

Figure 8. Stress-strain diagram of sample-4

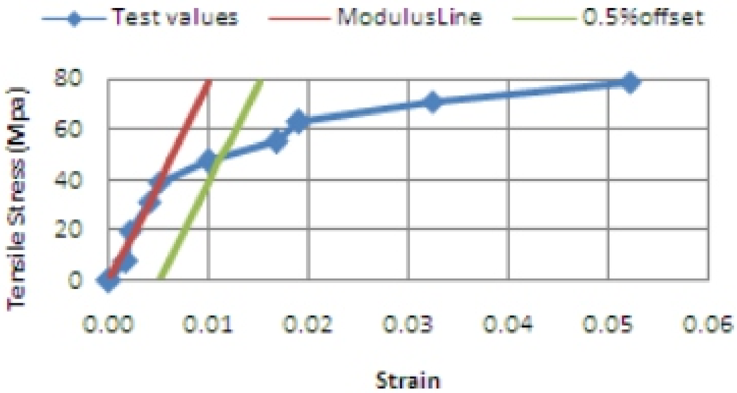

Figure 9. Stress-strain diagram of sample-5

Table 2. Yield Strength Ultimate Strength MoE of rattan (Zali)

Sample No	Yield Strength (MPa)	Average Yield Strength (fZ) (MPa)	Ultimate Strength (MPa)	Average Ultimate Strength (MPa)	Ratio of Ultimate and Yield Strength	MoE (MPa)	Average MoE (EZ) (MPa)
1	53	52	63	71	1.36	7486	8805
2	42		61			9004	
3	60		71			7925	
4	55		79			11686	
5	50		79			7922	

3.2. Result and Discussion on Flexural Test

Beams were prepared with various reinforcement ratios (ρ) using rattan at bottom as flexural reinforcement. The test result is shown at Figure 10. It is seen from the figure that as the reinforcement ratio (ρ) was increased the Crushing load (P) was also increased. The information related to crushing load to deflection of beam is presented at Figure 11. From the Load-deflection curves it is seen that continuously more deflection was exhibited by the Zali reinforced beam as the percentage of reinforcement ratio was increased.

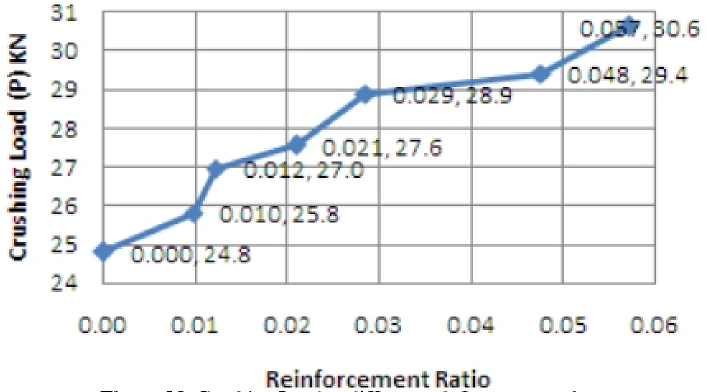

Figure 10. Crushing Load at different reinforcement ratio

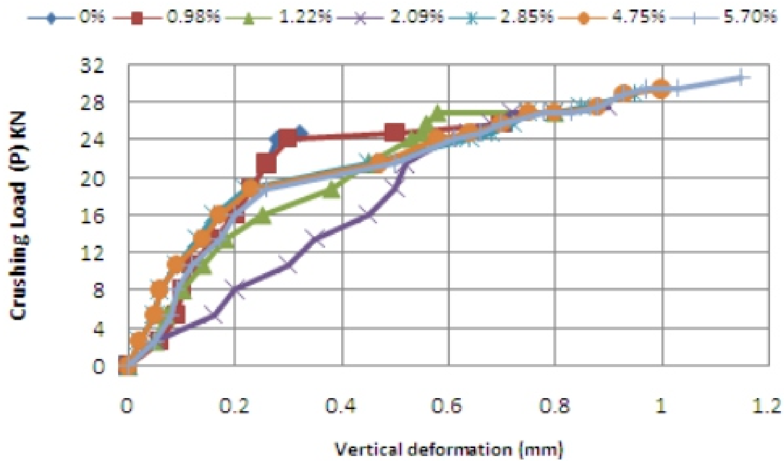

Figure 11. Load vs Deflection curves of Zali-reinforced beam at different reinforcement ratio

A number of cracks formed during the flexure test. All the cracks were purely flexural in nature. All of them were formed at middle of the beam, started at bottom and propagated vertically toward top. The cracks formed at different loads are of different beams are shown at Table-3.

Table 3. Detailed information on load, crack and deflection

Reinforcement Ratio	Load at 1st crack, KN	Load at 2nd crack, KN	Ultimate Load, KN	Maximum crack width, mm	Ultimate % increment of load	Central deflection at Ultimate Load, mm
0	24.8	---	24.8	---	0	0.32
0.0098	24	---	25.8	2.25	4	0.7
0.0122	24.5	---	26.96	3	8.7	0.8
0.0209	25	---	27.56	2.5	11.1	0.9
0.0285	25.5	27	28.88	2.5	16.4	0.95
0.0475	25.5	28.5	29.4	3.25	18.5	1
0.057	26	29.5	30.63	3.5	23.5	1.15

It was seen from the test that the non-reinforced concrete beam crushed as 24.8 KN load applied by UTM. Significant increase of load taking ability is shown by the beam where the rattan-Zali was used as flexural reinforcement. It was observed in the test that at the reinforcement ratio 0.0475, 29.4 KN load was taken by Zali reinforced concrete beam, which is 18.5% ($=\frac{(29.4-24.8)\times 100}{24.8}$) more than that of non-reinforced beam. The central deformation that was detected by strain gauge was 0.32 mm and 1.0 mm respectively. Linear shape of the Load-deflection curve was observed as expected for the non-reinforced beam. The beam cracked where the first crack formed. While for Zali reinforced concrete beam the first crack was observed at 25.5 KN. The corresponding central deflection was 0.65 mm that is much more than non-reinforced beam. After the 1st crack yet the beam was able to take load which indicates the active contribution of Zali. The 2nd (also final) crack was formed at 28.5 KN. For all beams load in UTM was measured up to the ultimate capacity.

For steel the equation of balanced ratio is shown in Equation 1 with usual notations. Since, the maximum strain in compression ϵ_u=0.003, strain of reinforcement ϵ_z= f_z / E_z, E_z =8805 MPa, so the modified equation for rattan (Zali) is shown in Equation 2. For rattan considering f_z = 52 MPa and $\beta_1 = 0.85$, using Equation 2, balanced reinforcement ratio $\rho_{z} = 0.0825$. In this study the maximum reinforcement ratio ρ is 0.057 that is lower than that of $\rho_{max,z}$. Therefore equations for single reinforced beams are taken to analyze the result. While the minimum reinforcement ratio $\rho_{min,z}$ is 0.0265. The detailed calculation is shown below. Based on the above discussion in present study the usable reinforcement ratios are 0.0285, 0.0475 and 0.0570.

$$\rho_b = 0.85\beta_1 \frac{f'_c}{f_y}\frac{\epsilon_u}{\epsilon_u+\epsilon_z} \ldots \qquad \ldots\ldots\ldots\ldots(1)$$

$$\rho_b = 0.85\beta_1 \frac{f'_c}{f_z}\frac{0.003}{0.003+\frac{f_z}{E_z}}$$

$$\rho_{z} = 0.85\beta_1 \frac{f'_c}{f_z}\frac{26.415}{26.415+f_z} \ldots\ldots \qquad \ldots\ldots\ldots(2)$$

$$\rho_{max,z} = 0.75\rho_{bz} = 0.75 \times 0.11 = 0.0825$$

$$\rho_{min,z} = \frac{1.38}{f_y} = \frac{1.38}{52} = 0.0265$$

Under inelastic analysis considering the beam section cracked the conventional moment equation is Equation [3]. The relation between the load (P) and moment (M) is shown in Equation [4] (Figure 1). Bond stress between concrete and reinforcement will be generated at the contact of them and the amount of bond force will be equal to the multiplication of bond stress and the surface area of reinforcement. A little consideration can expose that Equation [3] can be slightly modified to use the bond force as shown in Equation [5]. Since $A_z f_z$ (in Equation [3]) is the tension force taken by reinforcement therefore this force can be replaced by bond force. The lever arm of both equations [3] and [5] will be the

$$M_{the} = A_z f_z \left(d - \frac{a}{2}\right) \ldots \qquad \ldots.(3)$$

$$\text{where } a = \frac{A_z f_z}{0.85 f'_c b}$$

$$P = \frac{M_{eme}}{75} \ldots \qquad \ldots\ldots\ldots(4)$$

(where P in KN and M_{exp} in KN-mm)

$$M_{theo} = F\left(d - \frac{a}{2}\right) \qquad ......(5)$$

$$\text{where } a = \frac{F}{0.85 f'_c b}$$

Using the equations [3] and [4] the comparison of experimental and thetorical load is presented in Table 4. It seen that the difference between experimental and thetorical load ranges from 35% to 64%. Similarly, using the equations [4] and [5] the comparison of experimental and thetorical load is presented in Table 5. In this case that the difference between experimental and thetorical load ranges from 36% to -12%. Using flexure equation and bond stress the maximum reinforcement ratio is up to which the experimental values are less than the theoretical values are 0.017 and 0.048 respectively. The equation of bond stress has a closer position with the experimental values than that of equation of flexure (Figure 12). Therefore this is taken to predict the experimental values. The relation between reinforcement ratio (pp) and Pt BS is shown by Equation [6](Figure-12). Also relation between Pt BS and P_{exp} is shown by Equation [7](Figure-13). Since the allowable maximum reinforcement ratio is 0.0825 and the highest reinforcement ratio used in this study was 0.057 therefore prediction of load and moment was also made and presented at Table-6.

$$P_{t\,BS} = 540.9\rho + 3.52... \qquad ....(6)$$

$$P_{exp} = 0.163 P_{t\,BS} + 25.04 \qquad (7)$$

Table 4. Comparison of experimental and thetorical load considering section cracked

RF Ratio	Experimental		Theoretical						Diff P %
	P_{exp} KN	M_{exp} KN-mm	A_z mm^2	f_z MPa	D mm	A mm	M_{theo} KN-mm	P_{theo} KN	
0.029	28.88	2166	534.38	52	125	8.90	3349.84	44.66	-35
0.048	29.4	2205	890.63	52	125	14.83	5445.75	72.61	-60
0.057	30.63	2297	1068.75	52	125	17.79	6452.50	86.03	-64

Table 5. Comparison of experimental and thetorical load using Bond stress

RF Ratio	P_{exp} KN	No of bar	Avg Dia/bar mm	Length mm	Total perimeter mm	Surf Area mm^2	BS Mpa	F KN	a mm	M_{theo} KN-mm	P_{theo} KN	Diff P %
0.029	**28.88**	**4**	**13.0**	**150***	**163.85**	**24578**	**0.46**	**11.31**	**3.62**	**1392.75**	**18.57**	**36**
0.048	**29.4**	**6**	**13.8**	**150**	**259.07**	**38861**	**0.46**	**17.88**	**5.72**	**2183.33**	**29.11**	**1**
0.057	**30.63**	**7**	**13.9**	**150**	**306.54**	**45980**	**0.46**	**21.15**	**6.77**	**2572.27**	**34.30**	**-12**

*Note: Length=150=75+150/2

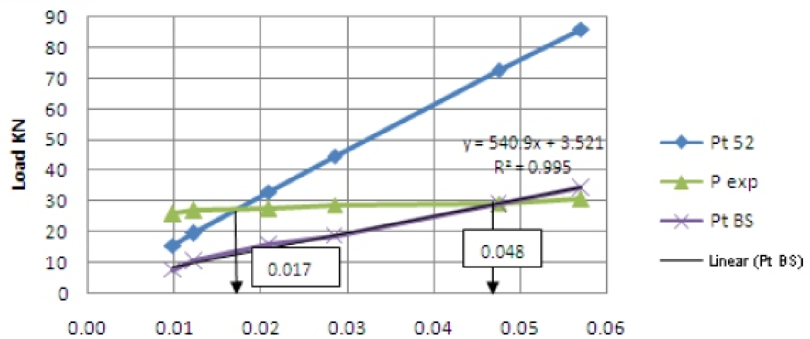

Figure 12. Relative position of Pe and Pt

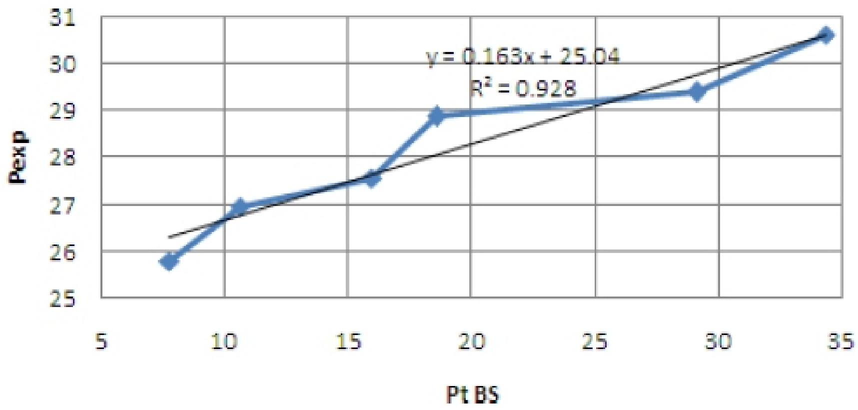

Figure 13. Relation between Pt BS and Pexp

Table 6. Predicted Pt BS, Pexpand Mexp at different reinforcement ratio

Reinforcement ratio	Pt BS Equ (6)	P_{exp} Equ (7)	M_{exp}(KN-mm) Equ (4)
0.06	35.98	30.90	2317.79
0.07	41.38	31.79	2383.92
0.08	46.79	32.67	2450.04
0.0825*	48.15	32.89	2466.58

*Maximum allowable reinforcement ratio in singly zali-reinforced beam

3.3. Result and Discussion Shear Test

The detailed information and test result about the beams for shear test is shown at Table 7. No flexural crack was observed in the beams, all the beams were failed showing inclined cracks as assumed. In all tests the load taking capacity of beams were increased as the spacing of the reinforcement was decreased. For example at spacing of 100 mm the load taking capacity of beam was increased 127% comparing the non-shear reinforced beam. The experimental results are analyzed by the conventional equation of shear (Equation 8) and bond stress. Here bond stress was taken 0.22 MPa since the GI wire was not used in shear reinforcement [7]. The theoretical crushing load using Equation (8) is shown Table 7. While using bond stress the theoretical crushing load is presented in Table 8. The relative comparison between the experimental and theoretical values are shown in Figure 13. It is seen from Table 7 and 8 that explanation using bond stress has closer position than that of the conventional equation of shear (Equation (7)).

Table 7. The detailed information of shear test result

Serial No	Reinforcement Type	Spacing, s mm	Cross-sectional area of Shear Reinforcement Av mm^2	Crushing load P_{exp} KN	% Improvement
1	Rattan-Zali	75	2* x 131.25	43.56	163
2	Rattan-Zali	100	2 x 131.25	37.63	127
3	Rattan-Zali	125	2 x 131.25	35.89	117
4	No Reinforcement	--	0.00	16.55	--

*Note: means two lagged

$$s = \frac{\phi A_v f_y d}{(V_{cr,d} - \phi V_c)} \tag{8}$$

$$=> V_{cr,d} = \phi V_c + \frac{\phi A_v f_y d}{s} = \frac{P}{2} \tag{9}$$

Table 8. Theoretical crushing load using Equation (7)

No	φ	$V_c = 0.17\sqrt{f_c'}bd$	A_v	Spacing S (mm)	$V_{cr,d}$(=P/2) KN	P KN	%Diff of P	Average Diff%
1	0.85	15.78	2 x 131.25	75	32.73	65.5	50	46
2	0.85	15.78	2 x 131.25	100	27.89	55.8	48	
3	0.85	15.78	2 x 131.25	125	24.99	50.0	39	

Table 9. Theoretical crushing load using bond stress

Bond stress Mpa	$v_c' = 0.17\sqrt{f_c'}$	V_c' Kip	Dia	No of bar	Force /bar	Total (2 lagged)	Pt BS KN	Diff P %	Average Diff%
0.22	0.84	15.78	12.9	4	1.11	8.91	24.7	-43	-42
0.22	0.84	15.78	12.9	3	1.11	6.68	22.5	-40	
0.22	0.84	15.78	12.9	2	1.11	4.46	20.2	-44	

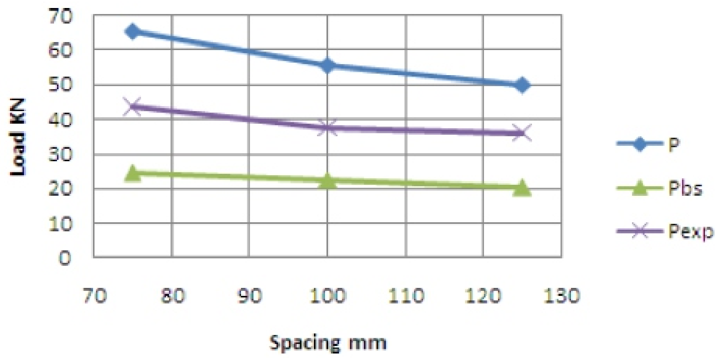

Figure 14. Ultimate load Vs spacing of shear reinforcement

3.4. Design Example

Studies were done by Dutta and Rose [3] and Uddin et al [10] using bamboo (local name Borak, and scientific name Bambosa balcoa) and rattan (local name Golla, and scientific name Demonorops jenkinsina) as reinforcement in concrete beam having the same experimental setup described in this paper. The yield strengths were 90 and 52 MPa respectively. The result regarding ultimate load against reinforcement ratio is shown in Table [10] and [11]. It is seen from the comparison that bamboo and golla reinforced beams have higher load taking ability than that of Zali-reinforced beam. And though the yield strengths of Golla and Zali are same yet the effectiveness of Golla was higher in beams as reinforcement.

Table 10. Ultimate load at different reinforcement (bamboo) ratio

ρ in Beam	Ultimate Load, KN
0.009	24.73
0.011	35.48
0.016	41.93
0.020	45.69
0.024	48.38
0.030	52.68
0.036	60.20

Table 11. Ultimate load at different reinforcement (Golla) ratio

ρ in Beam	Ultimate Load, KN
0.0122	25.5
0.0195	28.6
0.02.83	29.3
0.0320	32.3
0.0444	38.0
0.0556	41.3

Considering flexural capacity to visualize the effect of the above discussion a design is made to know the dimension (L in meter) of a square shaped room. Let a slab (75 mm thick) supported by four singly zali-reinforced simple supported single span beams. Also consider two live loads (LL) 2.0 KN/m2 and 0.0 concrete unit weight 23 KN/m3. The total loads in Ultimate Strength Design (USD) are, $1.2 \times \left(\frac{75}{1000} \times L^2 \times 23\right) + 1.6 \times (2 \times L^2) = 5.27L^2\ KN$ and $1.2 \times \left(\frac{75}{1000} \times L^2 \times 23\right) = 2.07L^2\ KN$.

Therefore each beam takes $w1 = \frac{5.27L^2}{4L} = 1.3175L$ KN/m and $w2 = \frac{2.07L^2}{4L} = 0.5175L$ KN/m.

Therefore the moment is shown by Equation (10). The detailed result is put on Table 12 where the experimental moment is taken from Table 4 and Table 6. After that the span (L) was calculated using Equation (10). Obviously in such case the cross-section of beam was considered same as shown by Figure 1.

$$M = \begin{cases} \frac{w_1 L^2}{8} = \frac{1.3175L^3}{8} \\ \frac{w_2 L^2}{8} = \frac{0.5175L^3}{8} \end{cases} \quad (10)$$

Table 12. Variation of span (L) with respect to Moment (flexural design)

RF Ratio	Moment KN-m	Span, L (m)	
		LL=2 KN/m^2	No LL
0.029	2.166	2.36	3.22
0.048	2.205	2.37	3.24
0.057	2.297	2.41	3.29
0.06	2.318	2.41	3.30
0.07	2.384	2.44	3.33
0.08	2.450	2.46	3.36
0.0825	2.467	2.47	3.37

Considering shear capacity another attempt can be made to visualize the capacity of rattan as shear reinforcement. Considering the above mentioned loading and geometric condition the equation of shear for is shown in Equation (11). The detailed result is put on Table 13 where the values of crushing load were taken from Table 7 and after that the span (L) was calculated using Equation (12). Obviously in such case the cross-section of beam is same as shown by Figure 3.

$$VV = \begin{cases} \frac{ww_1 l}{2} = \frac{1.3175 ll^2}{2} \\ \frac{ww_2 l}{2} = \frac{0.5175 ll^2}{2} \end{cases} \qquad (11)$$

$$PP = \begin{cases} 2VV = 1.3175 LL^2 \\ 2VV = 0.5175 LL^2 \end{cases} \qquad (12)$$

Table 13. Variation of span (L) with respect to crushing load (shear design)

Serial No	Spacing, s mm	Crushing load P(=2V) KN	Span, L (m)	
			LL=2 KN/m^2	No LL
1	75	43.56	5.75	9.17
2	100	37.63	5.34	8.53
3	125	35.89	5.22	8.33
4	No shear reinforcement	16.55	3.54	5.66

4. CONCLUSION

In this study evaluation was made on the use of a naturally available material- rattan (Zali) as both flexural and shear reinforcement in concrete beams. To understand the comparative scenario, beams having no reinforcement were also prepared. As a result it was seen that, the load taking capacity was upgraded by the use of rattan as reinforcement comparing to the non-reinforced beams. Since very few research was seen focusing on rattan as possible reinforcement in concrete, therefore the result of this research can provide the preliminary baseline for more research in this field.

The main concluding points of the study are:

a. Continuous increment of load taking ability was observed as the reinforcement ratio was increased in Flexure test. According to the test results the maximum 23.5% more load was taken by beams reinforced by rattan (Zali) comparing to non-reinforced beam (at a reinforcement ratio 0.057).

b. Continuous increment of load taking ability was observed as the spacing of reinforcement was decreased in Shear test. According to the test results the maximum 163% more load was taken by beams reinforced by rattan (Zali) comparing to non-reinforced beam.

c. The maximum and minimum reinforcement ratio was 0.0265 and 0.0825 respectively. In present study the maximum reinforcement ratio was 0.0570. Even at this ratio the placing of flexural reinforcement became too difficult to organize maintaining the same effective depth.

d. The result considering bond stress entailed closer position with the experimental results of flexure test than that of the conventional moment equation. Considering bond stress in analysis, 0.048 is the maximum reinforcement safe ratio up to which the thetorical values lies below the experimental value. After that though relations are made between PPt, BBBB,pp, mmmmdd, PPeemmee (in Equation (6) and (7)) yet the experimental results will be less than the theoretical values.

e. The result considering bond stress entailed lower position (42%) with the experimental results of shear test. The result considering shear equation entailed higher position (46%) with the experimental results of shear test.

f. The longevity of Zali as reinforcement was not studied in this paper. But for low load bearing and non important structures such reinforcement may be an effective alternative. Design made at section 3.4 showed the possible dimensions of a single story square shaped room with slab supported by the beams reinforced by rattan (Zali) considering two types of live load. The results showed that for small rooms such reinforcement may be used as an alternative of conventional reinforcement.

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