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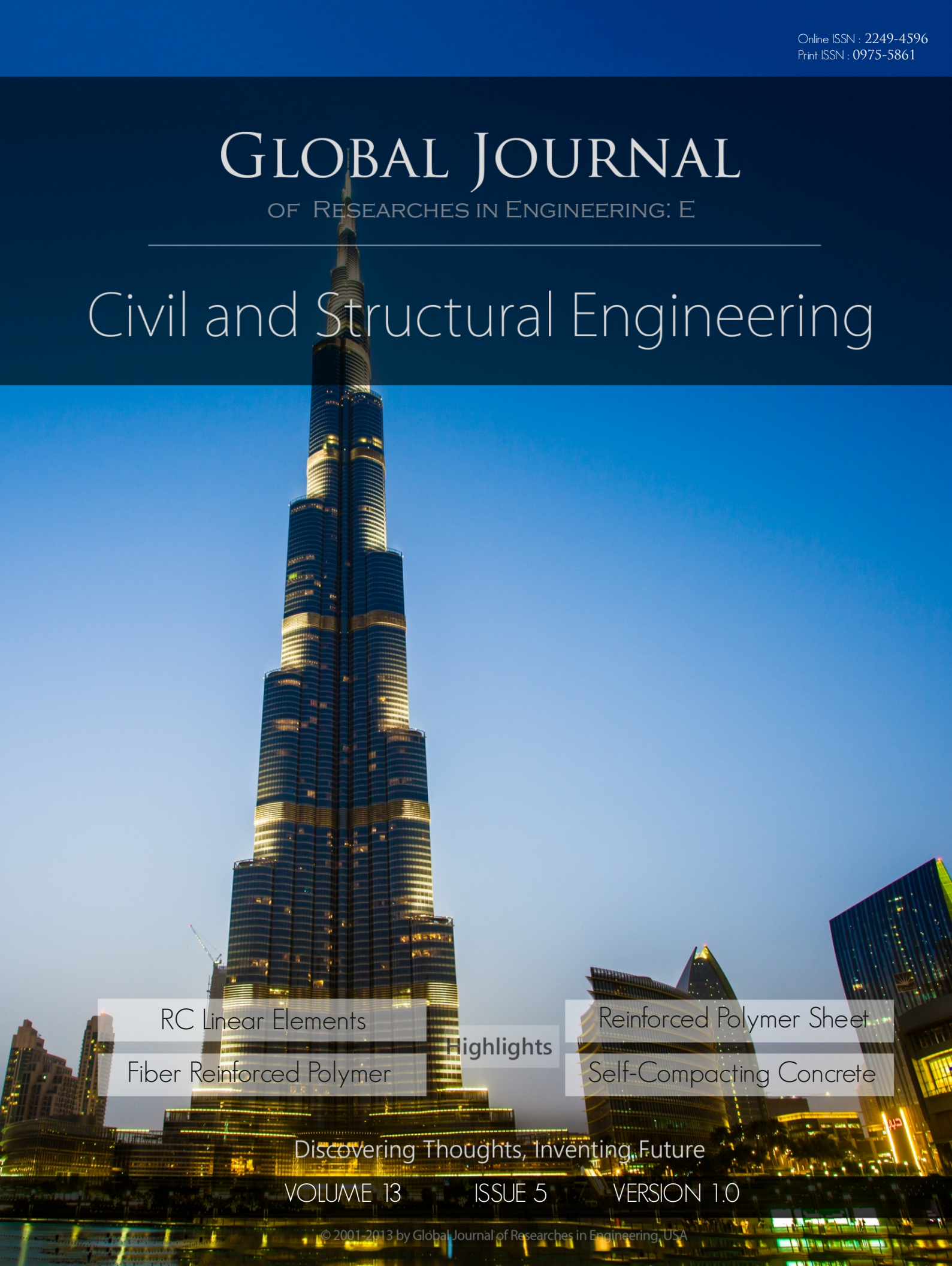

RC Linear Elements

Fiber Reinforced Polymer

Highlights

Reinforced Polymer Sheet

Self-Compacting Concrete

Discovering Thoughts, Inventing Future

VOLUME 13 ISSUE 5 VERSION 1.0

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Study of Sorptivity of Self-Compacting Concrete with Different Chemical Admixtures

By Saeed Alsheikh
MTI University- Cairo, Egypt

Abstract - The influence of chemical admixtures on the properties of Self-Compacting Concrete (SCC) was investigated. All types of used admixtures were the same percentage of 1.4% according technical data. The water – cement ratio was maintained at 0.36 for all mixes [8] [9].

The paper presents test results for acceptance characteristics of flow ability, resistance against segregation, and passing ability of self-compacting concrete in fresh state. Further, mechanical properties of hardened concrete such as compressive, tensile and flexural strength at the ages of 7 and 28 were also determined, and results of Absorption and sorptivity result are included here.

The results indicate that Sika ViscoCrete 3425 and AddiCrete BVS 100 give better results for all tests.

Keywords : *self-compacting concrete; sorptivity; absorption; durability; superplasticizer.*

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Study of Sorptivity of Self-Compacting Concrete with Different Chemical Admixtures

Saeed Alsheikh

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Keywords : self-compacting concrete; sorptivity; absorption; durability; superplasticizer.

I. Introduction

Self-Compacting Concrete (SCC) not only increases the reliability of structures but also reduces the number of workers required at the construction site and streamlines the construction. In pre-cast product plants as well, Self-Compacting Concrete is highly effective in reducing the noise as it requires no vibration [6]. SCC is a highly flowable, yet stable concrete that can spread readily into place and fill the formwork without any consolidation and without undergoing any significant separation. In general, SCC results in reduced construction times and reduced noise pollution [7].

SCC is defined as concrete that is able to flow and consolidate under its own weight, completely fill the formwork even in the presence of dense reinforcement, whilst maintaining homogeneity and without the need for any additional compaction [2]. Super plasticizer enhances deformability and with the reduction of wate / powder segregation resistance is increased [10] [11].

Sorptivity, which is an index of moisture transport into unsaturated specimens, has been recognised as an important index of concrete durability, because the test method used for its determination reflects the way that most concretes will be penetrated by water and other injurious agents and it is an especially good measure of the quality of near surface concrete, which governs durability related to reinforcement corrosion [12]. The sorptivity coefficient is essential to predict the service life of concrete as a structural and to improve its performance [13]. It was reported that the sorptivity of air-cured fly ash concrete, cured for 28, 90 and 180 days, increases with increase in fly ash content. In normal concrete has been shown that the condensed silica fume, under normal curing environments, to both increase strength and reduce sorptivity [14].

II. Experimental Investigation

a) Properties of Materials

i. Ordinary Portland cement (O.P.C)

Ordinary Portland cement (CEM I 42.5N) was used. Its typical physical properties and chemical analysis are shown in Table (1). The cement content was 500 kg/m3.

Table 1 : Properties of used Portland Cement

Description	Value
Physical Properties	
1- Specific gravity	3.15
2- Fineness passing 90 µm%	93%
3- Surface area cm^2/gm	3315
Chemical Analysis	
1- Lime Calcium Oxide (CaO)	60 : 67 %
2- Silicon Dioxide (SiO_2)	17 : 25 %
3- Aluminum Oxide (Al_2O_3)	3.0 : 8.0 %
4- Calcium Sulphate (CaSO)	0.50 : 6.0 %
5- Magnesium Oxide (MgO)	0.10 : 4.0 %
6- Sulphur trioxide (SO_3)	2.75 %
7- Alkalis	0.40 : 1.25 %
8- Loss in ignition %	3 %
Compressive Strength (Cubes)	
1- Age 2 days MPa	20.2
2- Age 7 days MPa	32.9
3- Age 28 days MPa	44.7

ii. Fine Aggregates -

Natural sand with medium size was used as a fine aggregate. Its physical properties were tested as specific gravity of 2.65 t/m3, fineness modulus of 3.65, absorption of 1%, unit weight of 1.68 t/m3, and voids ratio 31.7%. Sieve analysis had been conducted which its results are shown in Table (2).

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Table 2 : Sieve Analysis of Sand

Sieve Size (mm)	40	20	10	5	2.5	1.25	0.61	0.31	0.15
% Passing	100	100	100	90	70	50	20	5	0

iii. *Coarse Aggregates*

Dolomite of 15 mm maximum size was used. Its physical properties were tested as specific gravity of 2.72 t/m3, fineness modulus of 6.66, absorption 1%, the surface area of 2.06 cm2/gm, and crushing factor is equal to 12.50 %. Sieve analysis had been conducted which its results are shown in Table (3).

Table 3 : Sieve Analysis of Dolomite

Sieve Size (mm)	0	20	10	5	2.5	1.25	0.61	0.31	0.15
% Passing	99	95	35	5	0	0	0	0	0

iv. *Chemical Admixtures*

Four commercial products were used, they comply with ASTM C494-90 type "G" [3]and EN 934-2 [4]:

- AddiCrete BVS 100: Is Aqueous dispersion of modified polycarboxylate materials, it
- appears as brown liquid with specific gravity of 1.175 and solid content of 42.5%.
- Addi Crete BV 200: Is Aqueous dispersion of modified polycarboxylate materials, it
- appears as brown liquid with specific gravity of 1.11 and solid content of 27%.
- Sika Visco Crete 3425: Is Aqueous solution of modified polycarboxylate materials, it
- appears as clear liquid with specific gravity of 1.05 and solid content of 40%.
- Sika Visco Crete 5930: Is Aqueous solution of modified polycarboxylate materials, it
- appears as turbid liquid with specific gravity of 1.08.

b) Mixing procedure and moulding

The coarse and fine aggregates were initially fed into the concrete mixer, and then Portland cement and 3/4 of (water + admixture) were poured into the mixer. While the mixer was operated, the remaining water was added as necessary. The mixing time was 5.0 minutes started from the time when all the mixed materials had been charged into the mixer.

After casting, all the moulded specimens were covered with plastic sheets and were left in the casting room for 24 hours "25oC and 75 % R.H. Afterwards, they were de-moulded and transferred to the moist curing room at 100% relative humidity until required for testing.

c) Concrete Mixtures

An experimental program was undertaken to obtain workability, strength and durability for all mixes. Five mixes were made in this paper. For all mixtures, the graded coarse and fine aggregates were weighted in room dry condition, the coarse aggregate was then immersed in water for 24 hours, the excess water was decanted and the water retained by the aggregates was determined by the mass difference. A predetermined amount of water was added to the fine aggregate that was then allowed to stand for 24 hours. The water to cement ratio was maintained at 36%, coarse aggregate content (dolomite) was 875 kg/m3 with 15 mm, fine aggregate content (natural sand) was 950 kg/m3, tap water has been used for mixing and curing, tap water that used in all of the tests was clean drinking fresh water from impurities. Portland cement was used; the quantity of cement was 500 kg/m3. The mixture proportions of the mixtures are as shown in Table (4).

Table 4 : The Mix Proportion of Specimens

Mix No	w/c	Quantities kg/m^3					
		Cement	Water	Dolomite	Sand	Admixtures	
C	0.36	500	180	875	950	===	
M_1	0.36	500	180	875	950	7	AddiCrete BVS 100
M_2	0.36	500	180	875	950	7	AddiCrete BV 200
M_3	0.36	500	180	875	950	7	Sika ViscoCrete 3425
M	0.36	500	180	875	950	7	Sika ViscoCrete 5930

d) Test Method

i. *Fresh Concrete*

Self-Compacting Concrete is characterized by flow ability, passing ability and segregation resistance. Many different methods have been developed to characterize the properties of SCC. No single method has been found until date, which characterizes all the relevant workability aspects, and hence, each mix has been tested by more than one test method for the different workability parameters. Table (5) gives the recommended values for different tests given for mix to be characterized as SCC mix.

Table 5 : The Range of Values of Workability Tests

	Test	Characteristic	Typical range of values [1]	
			Minimum	Maximum
1	Slump Flow Diameter	Flow ability	650 mm	800 mm
2	L-Box (H_2/H_1)	Passing ability	0.8	1
3	V-funnel	Flow ability	6 sec	12 sec
	V-funnel at T_{5min}(time increase)	Segregation resistance	0 sec	3 sec

a. *Slump Flow Diameter*

It is the most commonly used test, and gives a good assessment of flow ability. The slump cone was filled with concrete without tamping; the concrete level at top of the cone was strike off. The cone was raised vertically to allow the concrete to flow out freely. Final diameter of the concrete in two perpendicular directions was measured. The average of the two measured diameters is the slump flow in mm. The apparatus is shown in fig. (1).

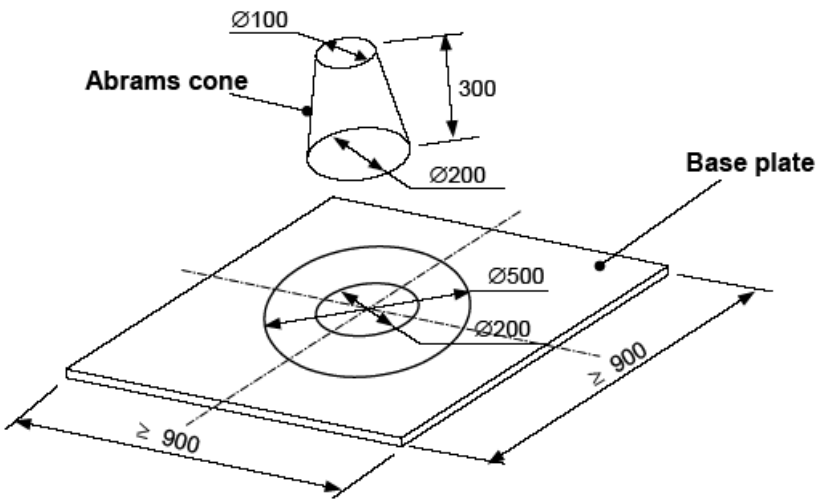

Figure 1 : Slump Flow Test

b. *L-Box*

It is a widely used test, and gives a good assessment of passing ability. The vertical section of apparatus was filled with concrete without tamping till level at top. After 1 minute; the sliding gate raised vertically to allow the concrete to flow out into horizontal section freely.

When the concrete stopped flowing, the heights of the concrete were measured; H1 in the vertical section, and H2 at the end of the horizontal section. The ratio of (H2/H1) is the blocking ratio. The apparatus is shown in fig. (2).

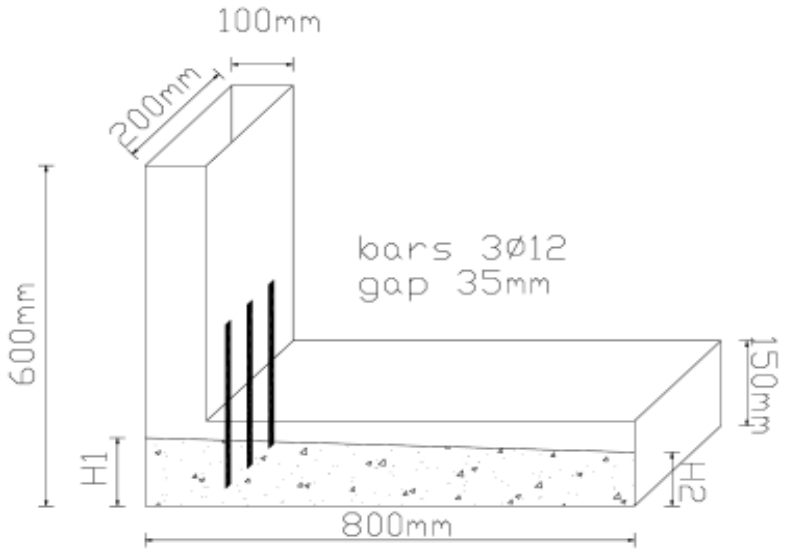

Figure 2 : L-Box Test

c. *V-funnel and V-funnel at T=5 minutes*

It gives a good assessment of segregation resistance. The funnel was filled with concrete without tamping; the concrete level at top of the funnel was strike off. The time was recorded when the trap door was opened to allow the concrete to flow out freely under gravity, till light is seen from above through the funnel. Immediately, the funnel refilled with the same concrete and left for 5 minutes to settle; then the time was recorded again. Shorter flow time indicates greater flow ability. The apparatus is shown in fig. (3).

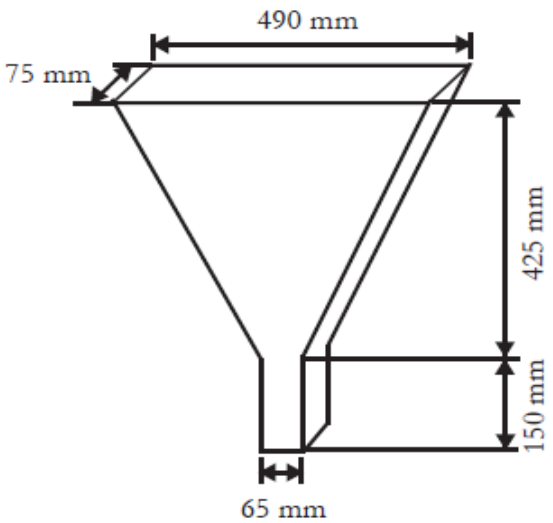

Figure 3 : V-funnel test

ii. *Hardened Concrete*

Self-Compacting Concrete is characterized by strength and durability. For strength requirements compressive, tensile, and flexural strengths were determined. For durability requirements water absorption and sorptivity were determined.

a. *Water Absorption*

The cubes of size 150×150×150 mm were used to determine the absorption at age of 28 days. The specimens dried in oven at temperature 105oC until the weight became constant, this weight was noted as dry weight (Wd). Then the cubes were immersed in water for 3 days then weighted, this weight was noted as wet weight (Ww). The %Absorption was computed by

$$\%Absorption = \frac{W_w - W_d}{W_d} \times 100 \quad (1)$$

b. *Sorptivity*

Sorptivity measures the rate of penetration of water into the pores in concrete by capillarity suction when the cumulative volume of water that has penetrated per unit surface area of exposure is plotted against the square root of time of exposure. The resulting graph could be approximated by a straight line passing through the origin. The slope of this straight line is considered as a measure of rate of movement of water through the capillary pores.

The cubes of size 150×150×150 mm were used to determine the sorptivity at age of 28 days [5]. The specimens dried in oven at temperature 105oC then side surfaces were sealed, and the end of the specimens opposite the absorbing surface was covered to impede evaporation from this surface during the test.

The specimens were supported on rods that it was submerged about 5 mm as shown in fig (4).

The sorptivity was computed by

$$S = \frac{}{A * \sqrt{t}} mm / \min^{0.5} \quad (2)$$

S = Sorptivity in (mm/min0.5)

Q = Volume of water penetrated in (mm3)

A = Surface area in (mm2)

t = time elapsed in (min)

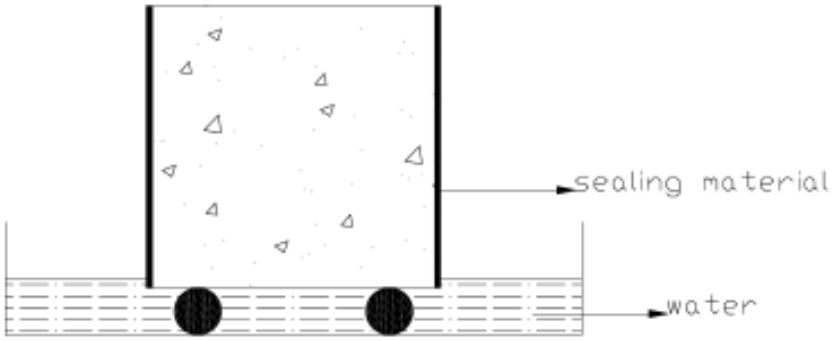

Figure 4 : Sorptivity test

III. Results and Discussion

a) Properties of Fresh Concrete

Concrete mixes at fresh state were tested as slump flow diameter, L-box and V-funnel, table (6) provides an overview of test results. Figures (5-7) provide a comparison of different tests for concrete mixes.

Table 6 : Tests Results At Fresh State

Mix. No.	Slump Flow Diameter (mm)	L-Box (H_2/H_1)	V-funnel (sec)	V-funnel at T_{5min} (sec)
C	675	0.80	10	12.5
M1	690	0.90	10.5	12
M2	654	0.86	11	14
M3	681	0.90	11	13
M4	670	0.83	7	8

i. *Slump Flow Diameter*

The slump flow diameter test was carried out according to EFNARC. The results measured are shown in table (6). In general, the slump flow diameters of mixes are in the range of 654:690 mm. Figure (5) shows the different values for each mix.

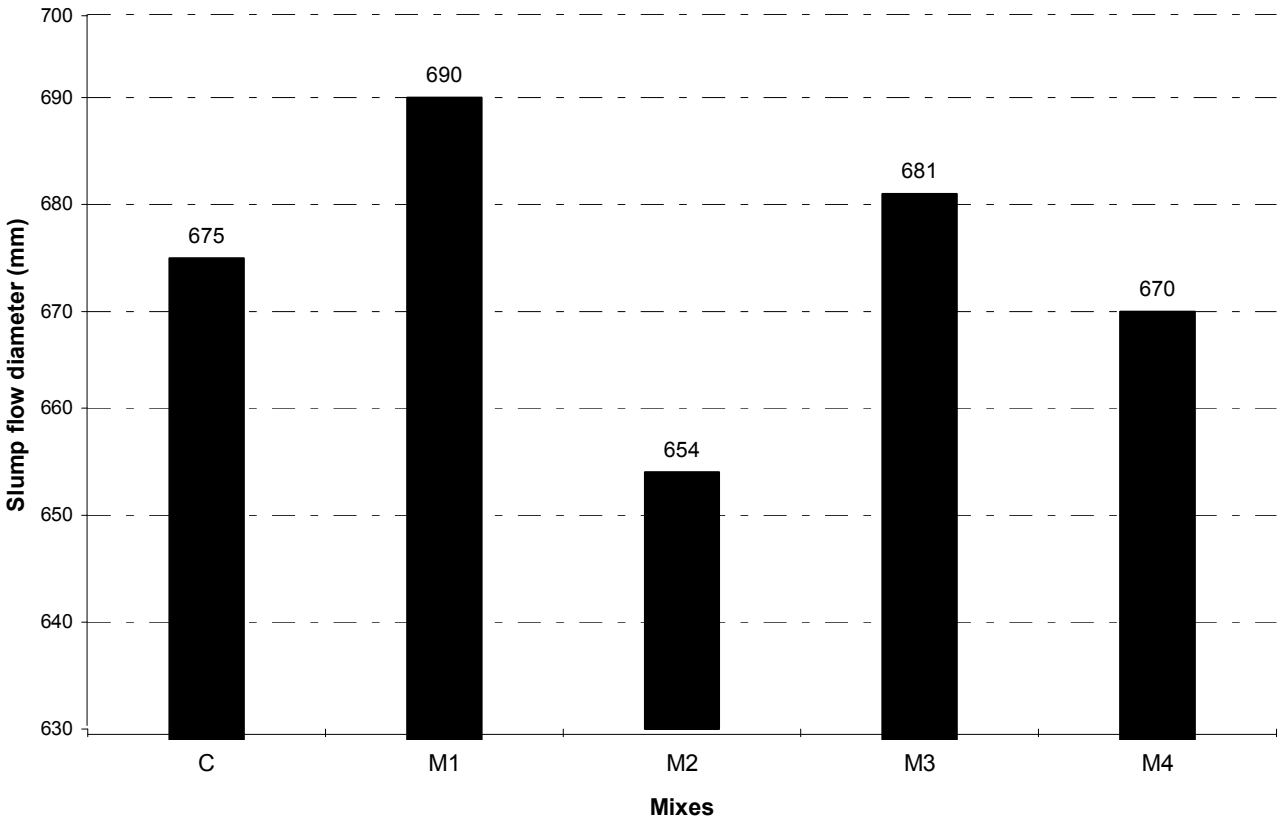

Figure 5 : Slump diameter test results

ii. *L-Box*

The L-Box test was carried out according to EFNARC. The results measured of blocking ratio are shown in table (6). In general, the blocking ratios of mixes are in the range of 0.8:0.9. Figure (6) shows the different values for each mix.

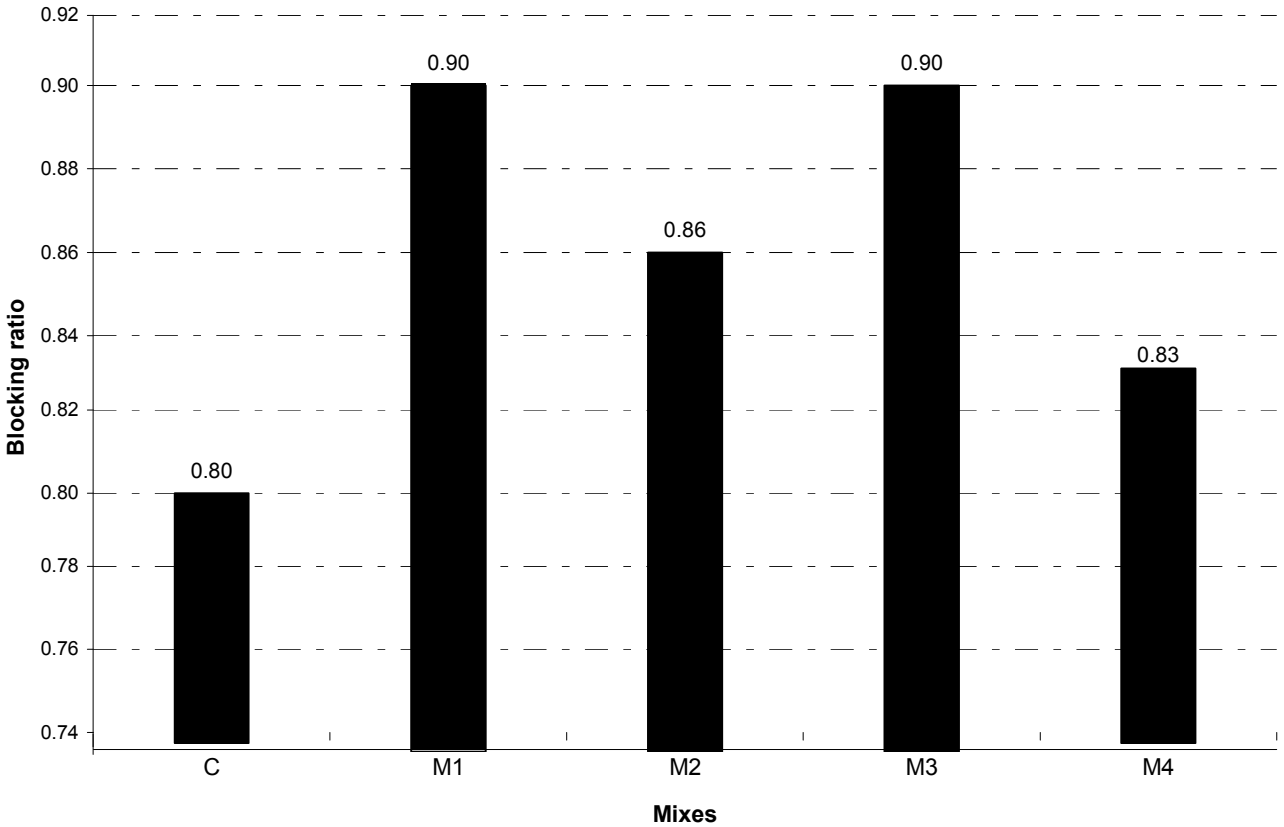

Figure 6 : L-box test results

iii. *V-funnel and V-funnel at T=5 minutes*

The V-funnel test was carried out according to EFNARC. The results measured of flow time are shown in table (6). In general, the flow times of mixes are in the range of 7:11 sec after 10 sec; and in the range of 8:14 sec after 5 minutes. Figure (7) shows the different values for each mix.

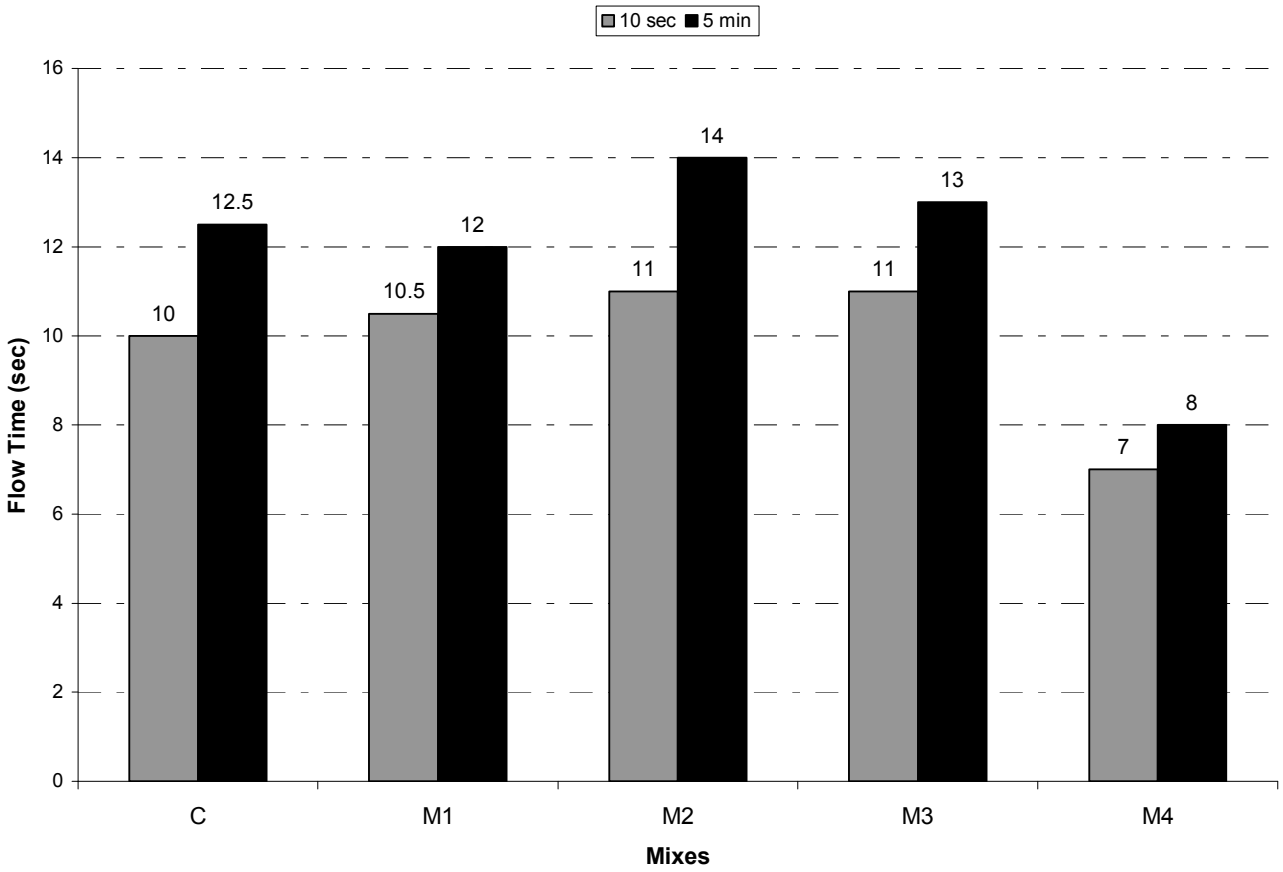

Figure 7 : V-funnel test results

b) Properties of Hardened Concrete

Concrete mixes at hardened state were tested as compressive strength, flexural strength and splitting tensile strength at different ages "7 & 28 days", Table (7) provides an overview of test results. Figures (8-12) provide a comparison of different tests for concrete mixes.

Table 7 : Tests Results at Hardened State

Mix. No.	Compressive Strength (kg/cm^2)		Tensile Strength (kg/cm^2)		Flexural Strength (kg/cm^2)	% Absorption	Sorptivity ($mm/\sqrt{min}$)
	7Days	28Days	7Days	28Days	28Days	28Days	28Days
C	350	445	25.38	28.21	36.3	5.495	0.2436
M1	315	458	36.14	42.22	53.0	5.307	0.1947
M2	149	212	27.83	30.67	30.5	8.433	0.2865
M3	437	506	28.97	31.56	50.3	4.020	0.1896
M4	319	369	21.98	24.06	42.5	6.688	0.3035

i. *Compressive Strength*

The compressive strength test carried out by ASTM C39. Its results are shown in Table (7). The compressive strengths for all mixtures are at range of 149 to 437 kg/cm2 after 7 days and at range of 212 to 506 kg/cm2 after 28 days. Figure (8) shows a comparison of achieved compressive strength for each mix.

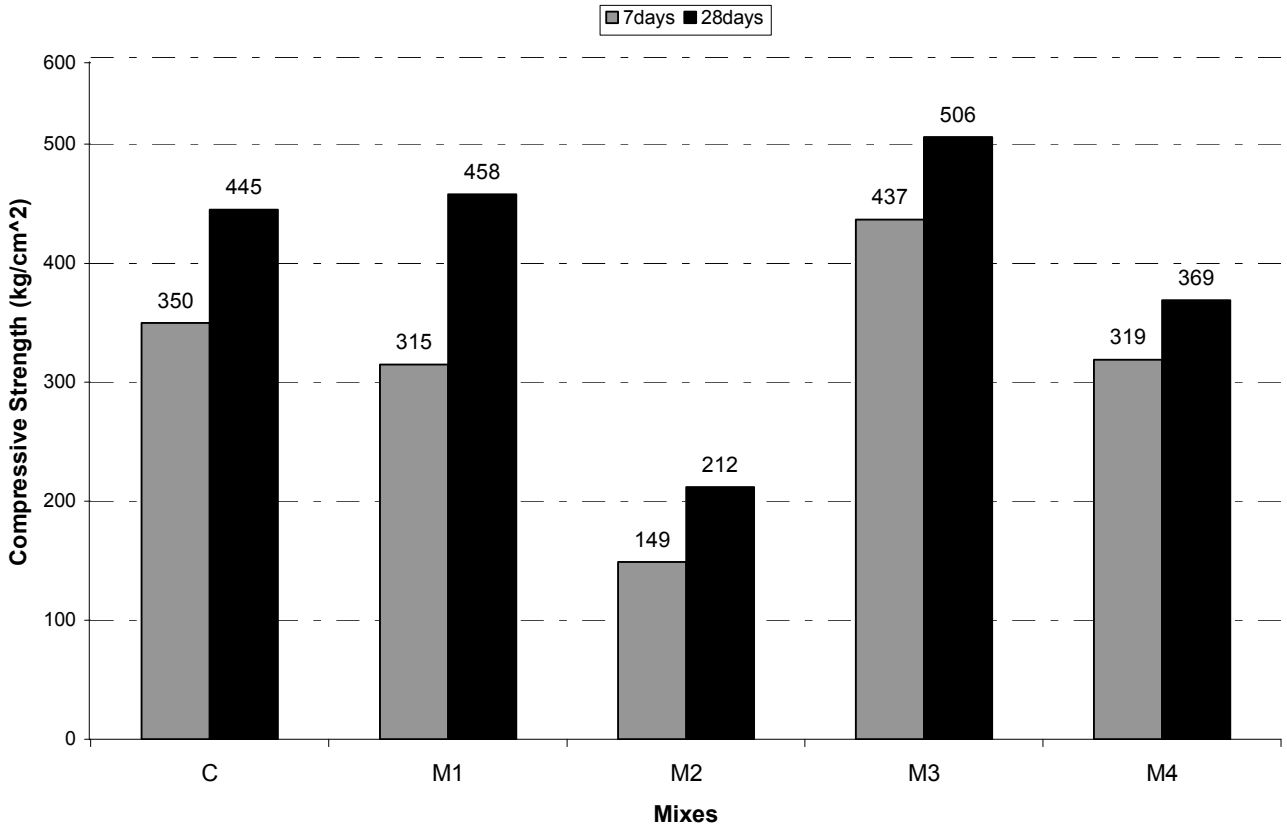

Figure 8 : Compression test results

ii. *Flexural Strength*

The flexural strength test carried out by ASTM C78. Its results are shown in Table (7). The flexural strengths for all mixtures are at range of 30.5 to 53.0 kg/cm2 after 28 days. Figure (9) shows a comparison of achieved flexural strength for each mix.

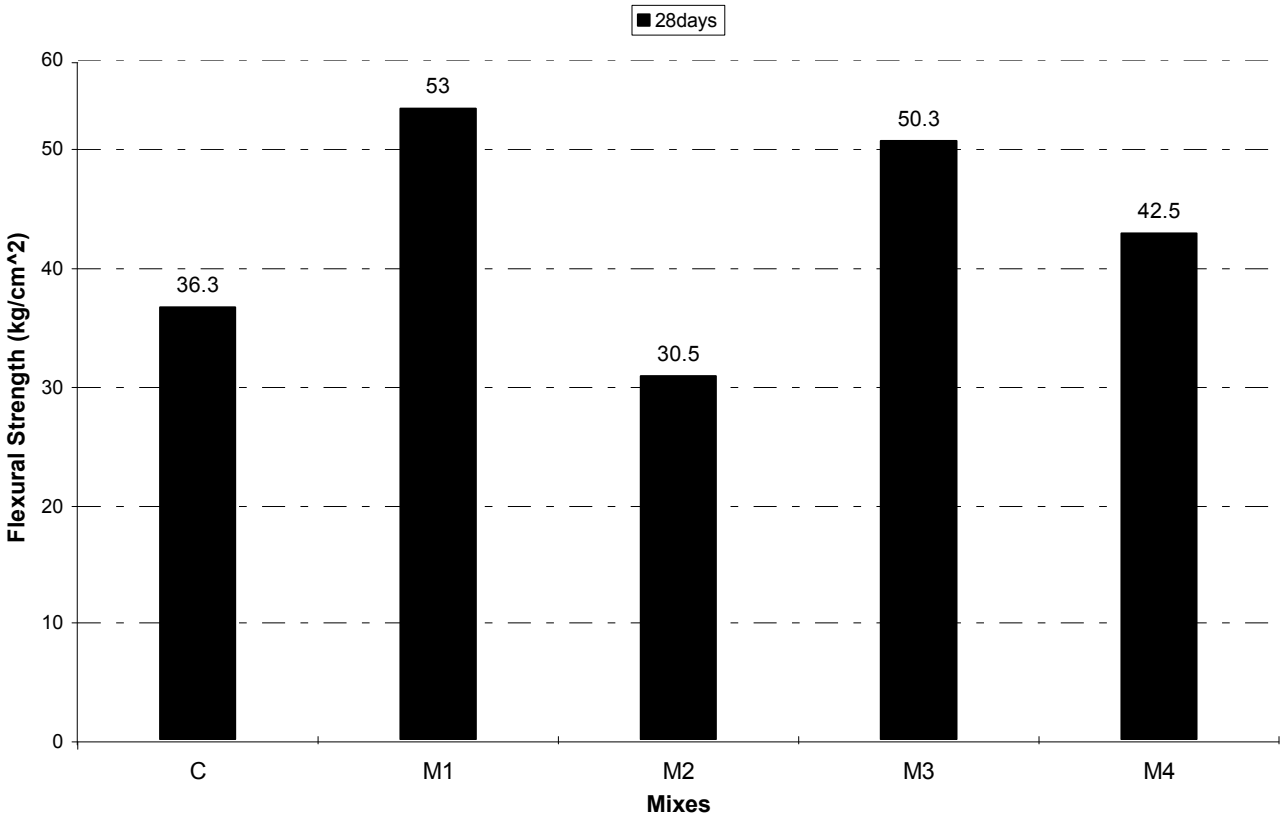

Figure 9 : Bending test results

iii. *Splitting Tensile Strength*

The tensile strength test carried out by ASTM C496. Its results are shown in Table (7). The tensile strengths for all mixtures are at range of 21.98 to 36.14 kg/cm2 after 7 days and at range of 24.06 to 42.22 kg/cm2 after 28 days. Figure (10) shows a comparison of achieved tensile strength for each mix.

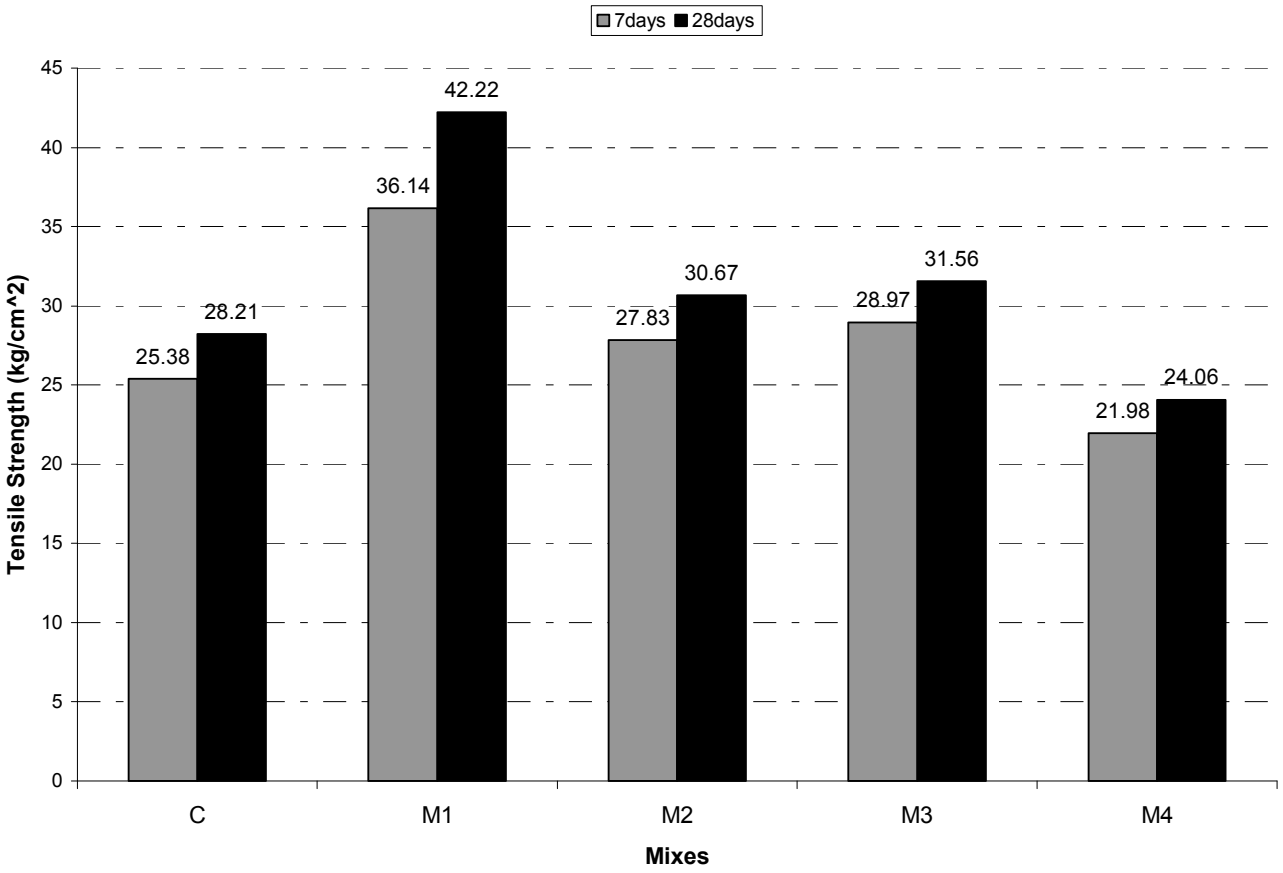

Figure 10 : Tension test results

iv. *Absorption*

The water absorption test carried out by ASTM C642. Its results are shown in Table (7). The water absorption percentages for all mixtures are at range of 4.02% to 8.433% after 28 days. Figure (11) shows a comparison of water absorption percentages for each mix.

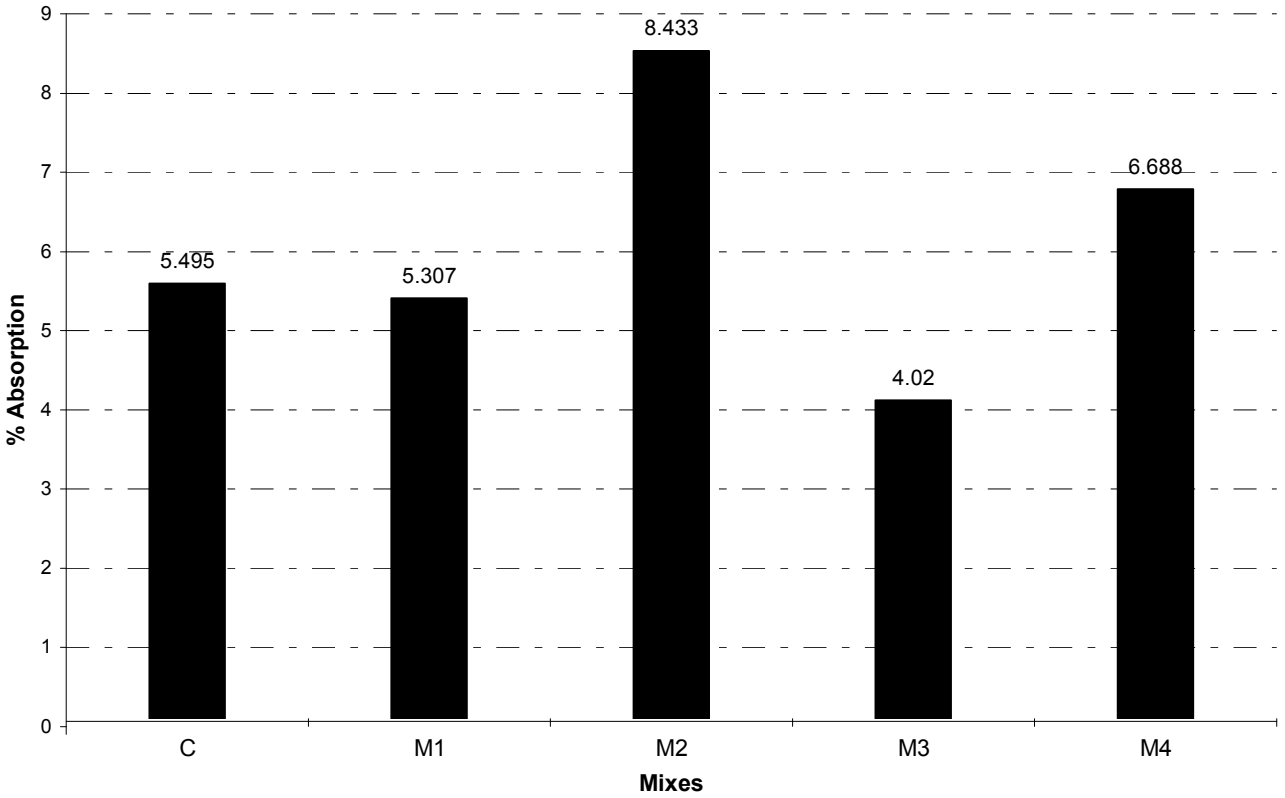

Figure 11 : Absorption test results

v. *Sorptivity*

The sorptivity test carried out by ASTM C1585. Its results are shown in Table (7). The sorptivity values for all mixtures are at range of 0.1896 to 0.3035 mm/√min after 28 days. Figure (12) shows a comparison of sorptivity values for each mix.

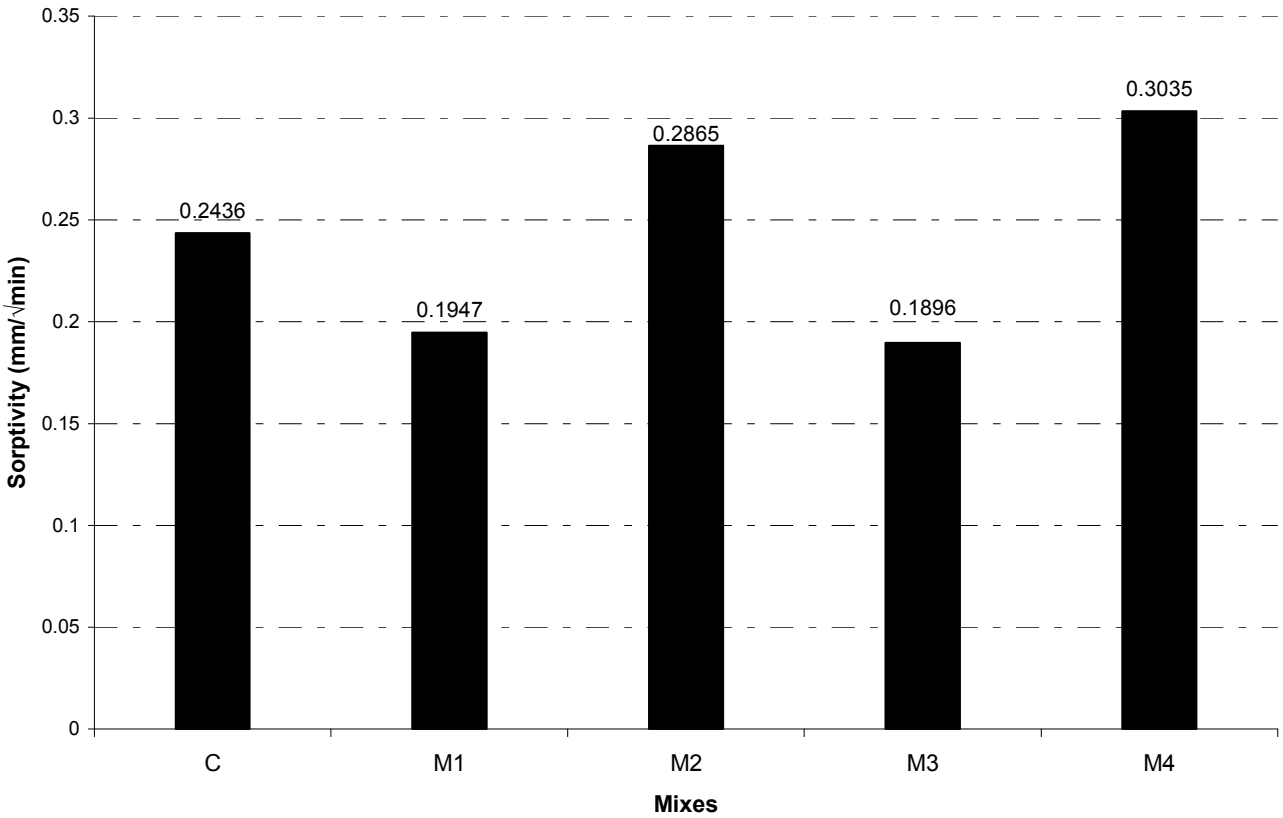

Figure 12 : Sorptivity test results

IV. Conclusions

Using Addi Crete BVS 100, Compressive strength decreased by (10.00%) at age of 7 days, and increased by (2.92%) at age of 28 days; Tensile strength increased by (42.40%) at age of 7 days, and increased by (49.66%) at age of 28 days; Flexural strength increased by (46.01%) at age of 28 days. % Absorption decreased by (3.42%) at age of 28 days, Sorptivity decreased by (20.07%) at age of 28 days. Flow ability increased, passing ability increased, segregation resistance.

Using Addi Crete BV 200, Compressive strength decreased by (57.43%) at age of 7 days, and decreased by (52.36%) at age of 28 days; Tensile strength increased by (9.65%) at age of 7 days, and increased by (8.72%) at age of 28 days; Flexural strength decreased by (15.98%) at age of 28 days. % Absorption increased by (53.47%) at age of 28 days, Sorptivity increased by (17.61%) at age of 28 days. Flow ability decreased, passing ability increased, segregation resistance.

Using Sika Visco Crete 3425, Compressive strength increased by (24.86%) at age of 7 days, and increased by (13.71%) at age of 28 days; Tensile strength increased by (14.14%) at age of 7 days, and increased by (11.88%) at age of 28 days; Flexural strength increased by (38.57%) at age of 28 days. % Absorption decreased by (26.84%) at age of 28 days, Sorptivity decreased by (22.17%) at age of 28 days. Flow ability increased, passing ability increased, segregation resistance.

Using Sika Visco Crete 5930, Compressive strength decreased by (8.86%) at age of 7 days, and decreased by (17.08%) at age of 28 days; Tensile strength decreased by (13.40%) at age of 7 days, and decreased by (14.71%) at age of 28 days; Flexural strength increased by (17.08%) at age of 28 days. % Absorption increased by (21.71%) at age of 28 days, Sorptivity increased by (24.59%) at age of 28 days. Flow ability decreased, passing ability increased, segregation resistance

From the previous investigation and test result, it can be concluded that Sika Visco Crete 3425 and Addi Crete BVS 100 improved the workability and strength properties of SCC.

Scope of the future work, the durability of concrete should be studied.

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Estimating the 3-Second Gust on Rooftops of Residential and Low-Rise Buildings during a Hurricane

By S. A. Hsu
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Abstract - After the passage of a hurricane numerous infrastructures suffer water and wind damages. From engineering meteorology viewpoint, many cases are related to the impact of the wind on a roof. In order to estimate the wind speed on a roof, the three second gust is employed according to ASCE-7 for the wind load analysis. However, since there is no 3-s gust measurement on a roof, constant disputes occur as to who is liable to pay for the damages. After a brief review of recent literature, this technical note provides a methodology to resolve these disputes objectively. The formula is verified by full-scale field measurements during Hurricanes Frances and Ivan. Furthermore, in order to help engineers and contractor estimate the 3-s gust on the rooftop, methods are provided so that the needed 3-s gust can be computed from wind speed measurement available routinely from airports.

Keywords : *wind speed on roofs; rooftop jet; gust factor; turbulence intensity; hurricanes.*

GJRE-E Classification : *FOR Code: 290804*

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Keywords : wind speed on roofs; rooftop jet; gust factor; turbulence intensity; hurricanes.

I. Introduction

During a tropical cyclone along hurricane-pronecoastal regions, many residential and low-rise buildings suffer structural damages, particularly their roofs. When the 3 second gust on a roof exceeds 33 m/s (74 mph), insurance companies should pay for these damages. However, many disputes occur between the insurance companies (representing the contractors) and the lawyers for the insurers because it is usually difficult to estimate the 3- s gust speed. The purpose of this technical note is to help engineers and contractorsso that estimation of3-sgust on roofs can be made objectively during hurricanes.

In the atmospheric boundary layer, the wind speed increases with height according to the power law such that

$$U_2 / U_1 = (Z_2/Z_1)^P \qquad Z_2 > Z_1 \qquad (1)$$

Where U_2 and U_1 are the wind speed at height Z_2 and Z_1, respectively, and P is the exponent.

Eq. (1) has been used widely inengineering community (see e.g. ASCE 7-02, in Irwin, 2006). The relationship between Eq. (1) and the more theoretically based logarithmic wind profile is provided in Hsu (1988). For detailed information relating the gust factor and the exponent P, see Hsu (2003 and 2008).

II. Relationship Between G and P

Since one minute sustained wind speed is used by the National Hurricane Center (see www.nhc. noaa. gov), from Hsu (2008), we have

$$G = 1 + 1.96\,P = 1 + 1.96\,TI \qquad (2a)$$

Where G is the gust factor (the ratio of 3-s gust to 1-min sustained wind Speed) and TI represents turbulence intensity.

Similarly, for 5 second gust over 2 minute period as used in wind speed measurements by the Automatic Surface Observing System (ASOS) station at airports worldwide,

$$G = 1 + 2.04\,P = 1 + 2.04\,TI \qquad (2b)$$

Where G is the gust factor (the ratio of 5-s gust to 2-min sustained wind speed) and TI represents turbulence intensity.

Further verification of G versus P is demonstrated in Fig. 1 for an industrial park (Crandell *et al.*, 2000). It is clear that a linear relationship between G and TI or P exists so that,

$$G - 1 = 2.13\,TI \qquad (3a)$$

Or

$$G = 1 + 2.13\,TI \qquad (3b)$$

R^2 = 0.91, indicating that 91% of the total variation of G can be explained by TI. Therefore, TI or P can be determined from the gust measurements such as at airports.

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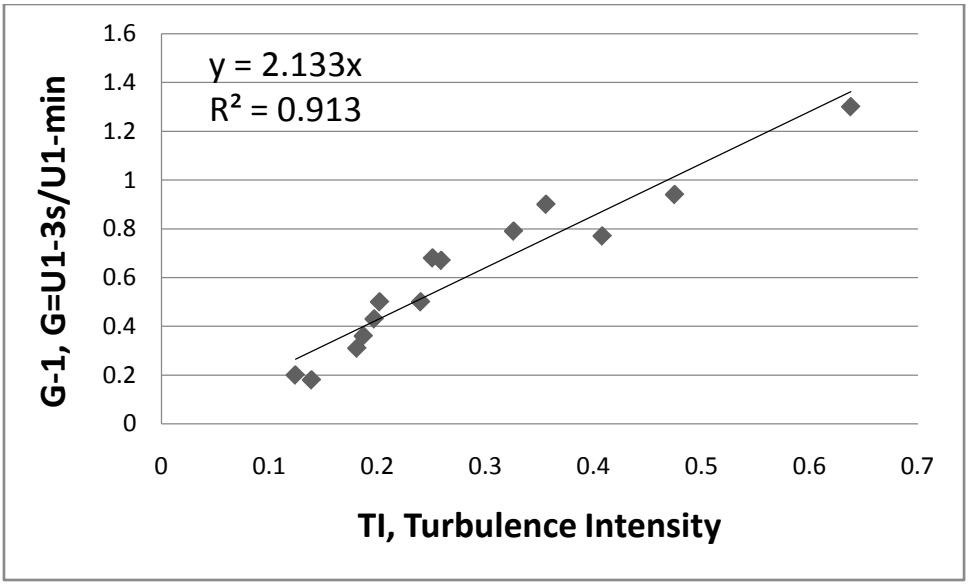

Figure 1 : Relationship between turbulence intensity and gust factor in an industrial park (Data source Crandell et al., 2000)

III. Review of Recent Literature

From engineering meteorology viewpoint, most cases dealing with buildings damaged by a tropical cyclone are related to the roofing system. In fact, during Hurricane Katrina in 2005, the roof of Louisiana Superdome in New Orleans, Louisiana, suffered severe damages. In addition, countless roofs in low-rise structures were destroyed by the gusty wind associated with Katrina.

Before cases are studied, some reviews of recent literature are helpful:

When air flows in a street canyon, Christen et al (2009) found that the highest turbulence intensity was located at 1.5 times the building height as depicted in Fig.2. Therefore, on the basis of Eq.(2a) and (2b), the gust factor is about two, meaning that the gust speed can be more than twice the sustained wind speed.

Because of this high gust speed we will name it as the rooftop jet. Note that at the rooftop edges and eaves, the gust factor can be 1.96 because TI = 0.48.

According to Aponte-Bermudez et al (2006), the average ratio of 3 second gust over one minute sustained wind speed for 10 residential houses in Florida during 5 hurricanes was 1.82 with a small standard deviation of 0.16. This means the coefficient of variation is only 9% (i.e. 0.16/1.82 = 0.09), which is within the 10% error margin for the wind measurements as stated previously.

Measurements and computations for flow and turbulence in simulated city canyon were investigated by Zajic et al (see www.geo.uni.lodz.pl/ ~icuc5/text /0_33_4.pdf). They found that the velocity U_h at the building height h correlates well with the wind speed U_r at the reference height Z_r for the first row of containers according to a power-law profile such that

$$U_h/U_r = (h/Z_r)^{0.53} \quad (4)$$

Finally, in an urban area such as New York City during Hurricane Irene in 2011, the gust factor from ASOS station at Central Park was 1.86 whereas in its vicinity at LaGuardia and Kennedy Airports, the gust factors are 1.29 and 1.28, respectively (see Avila and Cangialosi, 2011). Substituting these gust factors into Eq. (2b), we get TI = 0.42 for the Central Park and 0.14 for the two airports nearby. These values are consistent with literatures in that p = 0.40 for the urban area and 0.16 for the flat open country known for a long time (see Davenport, 1965). The TI (=0.42) is also in agreement with Fig.2, since the ASOS station in the Central Park is located near the top of many trees and some buildings.

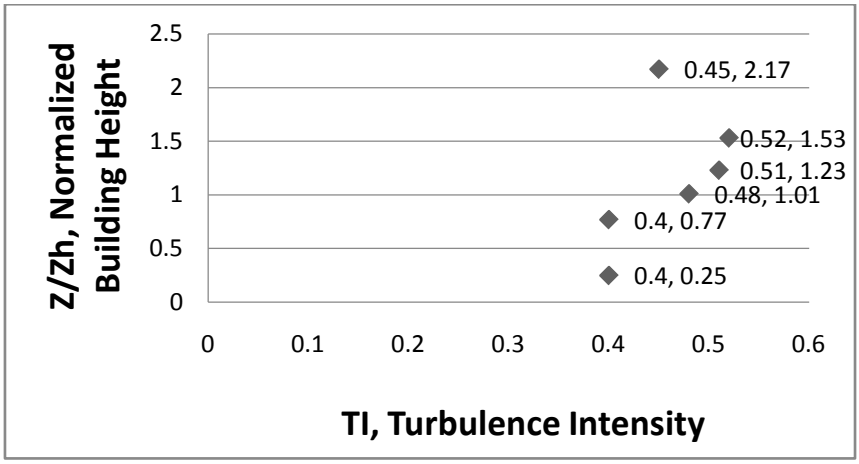

Figure 2 : Variations of the turbulence intensity with normalized building height (Data source: Christen et al., 2009)

IV. Full-Scale Measurements During Hurricanes

Full-scale measurements for hurricane wind loads on residential structures were made by Aponte-Bermudez et al (2006). Their results are summarized in Table 1.

Table 1 : Verification of Equation (4) Based on Full-Scale Measurements of Three Second Gust on Rooftops in Low-rise Residential Buildings during Hurricanes in 2004 (Data Source: Aponte-Bermudez et al., 2006)

Hurricane	House ID	City, State	House Anemometer Height, m	Rooftop Measurements, 3-second, m/s	Rooftop Jet From Eq.(4), 3-second, m/s	Estimated Open Exposure At 10m, 3-s, m/s
Frances	FL-06	Jensen Beach, FL	7.062	40.6	39.3	47.3
Ivan	FL-30	Pensacola, FL	6.553	48.7	40.7	50.9
	FL-27	Gulf Breeze, FL	6.553	36.6	34.6	43.3
	FL-26	Navarre, FL	6.096	26.3	30.5	39.7
	FL-24	Destin, FL	6.096	25	26.8	34.8
	FL-23	Destin, FL	6.096	25.4	25.8	33.5
			Mean	33.8	33.0	41.6

V. Objective Methodology

According to Marshall (see http://ams.confex.com/ams/ pdf papers/137547.pdf) , during Hurricane Katrina, there was an anemometer located at 1.5m above the southeast roof corner on a 16m high building at Ingalls Shipyard in Pascagoula, Mississippi, USA. That anemometer recorded 3-second wind gust up to 53 m/s. On the other hand, an anemometer located at nearby Trent Lott International Airport recorded a peak 3-second gust of 41.6 m/s at 10m. These precious data provide us an opportunity to verify Eq. (4) as follows:

By setting U_r = 41.6 m/s, h = 16 m (the roof height), and Z_r = 10m into Eq. (3), we have U_h = 53 m/s, which is the same as measured.

Further verification of Eq. (4) is provided in Table 1. Since the difference between the two means (i.e. 33.8 m/s as measured on rooftops versus 33.0 m/s as estimated by Eq. (4)) is only 2.4%, Eq. (4) is also very useful for estimating the 3-secondgust impacting on the rooftop by hurricanes. Note that the 3-s gust is needed to determine whether the insurance companies are liable to pay the home-owners. In U.S., if the 3-s gust exceeds 33 m/sor 74 miles per hour as Category One

Hurricane, the insurance company usually reimburses the home-owner to replace the roof damages.

On the basis of foregoing analyses, it is recommended that for the estimation of three second wind gust impacting on a building, Eq. (4) can be employed to determine the rooftop jet speed. This finding should be very useful for structural engineers as well as insurance industry, since constant disputes occur between the insurance companies and insurers. Now, we can apply Eq. (4) to determine objectively the magnitude of the 3-second gust on a rooftop using the 3-second gust measurement from ASOS station normally located at airports. If the 3-s gust is not available from ASOS, remedial methods are provided in the next section.

VI. Estimating the Three-second Gust from ASOS Measurements

Because the 3-second gust data is not always available, we need to estimate it from the 2-minute sustained wind speed, which is routinely measured and reported from ASOS station at airport. On the basis of simultaneous wind speed measurements between ASOS and Texas Tech stations during Hurricane Bonnie as provided in Table 2 (see Schroeder, 1999) we can get the needed 3-second gust by multiplying the 2-minute sustained wind speed by a factor of 1.38. This factor is obtained from the ratio of 33.6/24.4 = 1.38.

Table 2 : Comparison of nearly simultaneous measurements of wind speeds as obtained via Texas Tech University and ASOS Stations for Hurricane Bonnie (Data source: Schroeder, 1999). Unites are in m/s

	ASOS Station	Texas Tech Station
0.2-Second Gust	NA	38.2
3-Second Gust	NA	33.6
5-Second Gust	32.9	33.5
1-Minute Sustained	NA	25.0
2-Minute Sustained	25.2	24.4

VII. Transition of Terrain Exposure

According to ASCE-7, (see, e.g., Zhou and Kareem, 2002), the variation of wind profile due to different terrain exposures, e.g. urban versus rural must be taken into account. In this regard, methods provided in Zhou and Kareem (2002) can be employed.

VIII. Conclusions

On the basis of aforementioned analyses and discussions, we conclude that the three second gust impacting on residential or low-rise building roofs during a hurricane can be determined from Eq. (4) objectively. Methods are also provided that the needed 3-s gust can be computed from the routine wind measurements at the airports. These methods should help engineers and contractors resolve the disputes arising from damages caused by a hurricane. However, since the wind profile varies with the terrain exposure, adjustment of terrain transition needs to be taken into account

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Studies on Behaviour of Rcc Beam-Column Joint Retrofitted with Basalt Fiber Reinforced Polymer Sheet

By G. Maariappan & R. Singaravadivelan
Jeppiar Engineering College, India

Abstract - Reinforced Concrete (RC) buildings designed for IS 456-2000 have been found to be weak in adequate seismic design provisions, capacity design considerations and detailing for ductile behaviour. Experimental tests RC frames have shown that the excessive damage or failure of beam-column joints, in particular exterior (or corner) joints which can lead to the global collapse of a building. The poor joint behaviour of older construction can be attributed to the inadequate shear reinforcement in joint region and the deficient anchorage details into the joint region. Recent evaluation of Civil Engineering structures has demonstrated that most of them will need major repairs in the near future. Up gradation to higher seismic zones of several cities and towns in the country has also necessitated in evolving new retrofitting strategies.

One of the techniques of strengthening the RC structural members is through confinement with a composite enclosure. This external confinement of concrete by high strength fibre reinforced polymer (FRP) composites can significantly enhance the strength and ductility as well as result in large energy absorption capacity of structural members .FRP materials such as basalt, glass and hybrid fibre, available today in the form of sheets, are being used to strengthen a variety of RC elements to enhance the flexural, shear and axial load bearing capacity of elements.

Keywords : *basalt fiber, epoxy resin, cement, aggregate and water.*

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Keywords : basalt fiber, epoxy resin, cement, aggregate and water.

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I. Introduction

In the last few decades, moderate and severe earthquakes have struck different places in the world, causing severe damage to reinforced concrete (RC) structures.

Retrofitting of existing structures are the major challenges that modern civil engineering field is facing these days. Recent evaluation of civil engineering structures has demonstrated that most of them will need major repairs in the near future. Up gradation to higher seismic zones of several cities and towns in the country has also necessitated in evolving new retrofitting strategies.

In RC buildings, portions of columns that are common to beams at their intersections are called beam-column joints. Since their constituent materials have limited strengths, the joints have limited force carrying capacity. When forces larger than these are applied during earthquakes, joints are severely damaged.

Beam column joints in a reinforced concrete moment resisting frames are crucial zones for transfer of loads effectively between the connecting elements (i.e. beams and columns) in the structure. In normal design practice for gravity loads, the design check for joints is not critical and hence is not usually done. But, the failure of reinforced concrete frames during many earthquakes has demonstrated heavy distress due to shear in the joints that culminated in the collapse of the structure.

a) Objectives

In general this investigation was carried out to study the behaviour of the beam-column joint under static and reverse loading. In more specific terms this research was conducted to achieve the following objectives

- Studies and behaviour of reinforced concrete beam-column joint retrofitted with Basalt fibre reinforced polymer sheets (BFRP).

b) Literature Review

Dylmar Penteado Dias (2005)[1] et al The purpose of this work was to investigate the influence of the volumetric fraction of the fibers on the fracture

toughness of geopolymeric cement concretes reinforced with basalt fibers.

- T. Cziga´ny(2006)[2] The strength properties of hybrid composites improved owing to surface treatment and this was proven by mechanical tests and microscopic analysis, as well.
- Jongsung Sim,(2006)[3] et al This study investigates the applicability of the basalt fiber as a strengthening material for structural concrete members through various experimental works for durability, mechanical properties, and flexural strengthening.
- M.M. Smadi et al(2008)[4] Ten slab–column connections were tested under combinations of gravity and lateral loads to investigate the effect of adding steel fibers to concrete mix on the structural behavior of normal- and high-strength slab–column connections.
- Bu¨ lent O¨ ztu¨ rk (2006)[5] et al In the present study, hybrid friction materials were manufactured using ceramic and basalt fibers. Ceramic fiber content was kept constant at 10 vol% and basalt fiber content was changed between Experiments show that fiber content has a significant influence on the mechanical and tribological properties of the composites.
- Xin Wang et al (2010)[6] To overcome the limitations of conventional steel stay cables in a thousand-meter scale cable-stayed bridge, hybrid basalt and carbon (B/C) FRP cables were investigated to achieve integrated high performances in the bridge of this scale as a replacement for steel cables.
- Mohamed F.M. et al (2010)[7] Commonly used fiber-reinforced polymer (FRP) includes Carbon, Glass, and Aramid FRP composites. The aim of the study is twofold. In case of different types of FRP composites, providing equivalent confinement modulus (lateral stiffness), five models are employed to find the FRP-confined concrete stress–strain relationship of three scale-model circular columns.
- Catherine Papanicolaou , et al (2010)[8] Externally bonded grids are used in this study as a means of increasing the load-carrying and deformation capacity of unreinforced masonry (URM) walls subjected to cyclic loading.

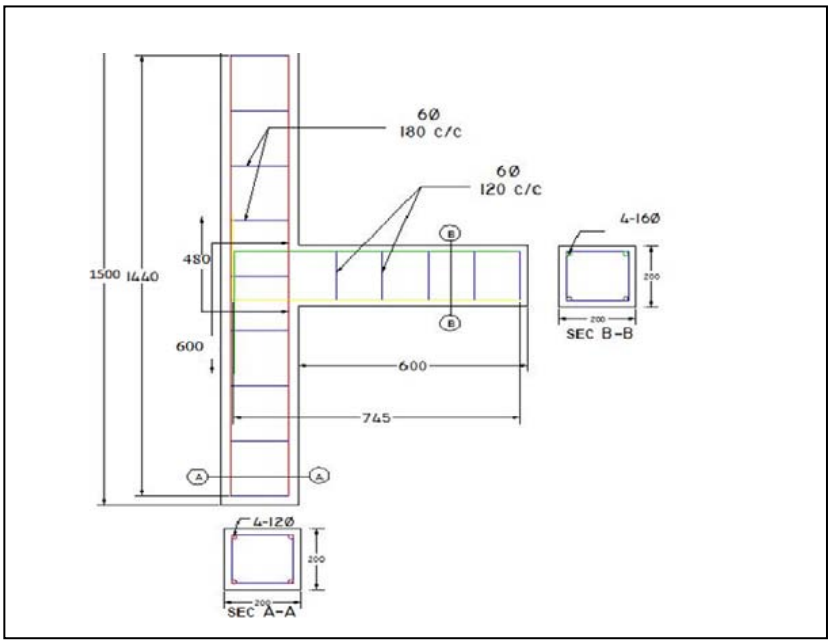
c) Experimental Investigation

The experimental program consisted of testing of nine reinforced concrete beam-column joint specimens. The columns had a cross section of 200 mm x 200 mm with an overall length of 1500 mm and the beams had a cross section of 200 mm x 200 mm with a cantilevered portion of length 600 mm. In fhree specimens, the lateral ties in the column are provided with spacing 180 mm c/c as per IS 456:2000. In remaining three specimens, the lateral ties in the column are provided with spacing 80 mm c/c and 100 mm c/c as per IS 13920:1993. The concrete mix was designed for a target strength of 30 MPa at the age of 28 days. The load carrying capacity of the column was evaluated as 525 kN as per the code IS 456-2000.

d) Parameters Investigated

- M30 grade concrete was made with a mix ratio of 1:1.502:2.558. Companion cubes were made to find the value of characteristic strength of concrete. The longitudinal reinforcement in the column portion in all the specimens consisted of 4 no. 12mm Ø (HYSD) bars. The tension reinforcement in the beam portion consisted of 2 no 16mm Ø bars and the beam compression reinforcement consisted of 2 no 16mm Ø bars. The anchorage length of the tension and the compression reinforcement of the beam is extended into the column as per codal provision. The details of the specimens are given below.

SINo	Name of Specimen	Code Reference	Criteria Axial Loads	Retrofitting
1	BCJ 1	IS 456	15%	Nil
2	BCJ 2	IS 456	30%	Nil
3	BCJ 3	IS 456	45%	Nil
4	BCJ 4	IS 13920	15%	Nil
5	BCJ 5	IS 13920	30%	Nil
6	BCJ 6	IS 13920	45%	Nil
7	BCJ R1	IS 456	15%	Basalt Fiber
8	BCJ R2	IS 456	30%	Basalt Fiber
9	BCJ R3	IS 456	45%	Basalt Fiber



e) Reinforcement Details of Beam Column Joint Specimen

i. *Preparation of Mould*

Moulds made of steel sheet had been welded and prepared for casting the beam column joint specimen. It consists of a long steel plate and two L-shaped welded plates and this assembly was bolted together by using square plates at the ends. The inner dimensions of the mould are 1500 x 200x 200 mm in the column portion and 600 x 200 x 200 mm in the beam portion.

ii. *Casting of Test Specimen*

The Reinforced concrete beam column joint specimens were cast using specially fabricated steel moulds. Two moulds were prepared for this purpose. The fabricated reinforcement steel was placed inside the mould and it is kept in position using cover blocks.

Concrete was mixed manually and poured into the moulds. Care was taken to see that the concrete was properly placed and compacted beneath and also on the sides of the mould using a needle vibrator.

The sides of the mould were removed after 24 hours from time of casting and the test specimens were cured for water using gunny bag coverings. 3 cubes of sizes 150 x 150 x 150 mm were cast along with each test specimen for evaluating the 28day compressive strength of concrete. Figure describes the above mentioned casting and curing operations.



iii. *Preparation of the Retrofitted Specimens*

The failed specimens BCJ 1, BCJ 2, and BCJ 3, were retrofitted and new specimens BCJR1, BCJR2 and BCJR3. The concrete near the area of failure was removed completely. After applying cement paste in this area, the portion was filled and compacted with the same grade of concrete. The specimens were cured for 28 days. Before wrapping the Basalt fiber sheet the faces of the specimens were ground mechanically to remove any laitance. All the voids were filled with putty. Then a two component primer system was applied on the concrete surface and allowed to cure for 24 hours. A two component epoxy coating was then applied on the primer coated surface and the Basalt fiber sheet was immediately wrapped over the entire surface of the reinforced concrete beam-column joint.

A roller was then applied gently over the wrap so that good adhesion was achieved between the concrete surface and the Basalt fiber wrap, as suggested by the manufacturers and allowed to cure for seven days. Another coat of the two component epoxy was applied over the fiber sheet. Then the second wrap was applied by following the same procedure and allowed to cure for a further period of seven days. Both the wrapped layers were orthogonal to each other.

iv. *Description of the Test Program*

The RC beam-column joint specimens were tested using loading frame in the structural laboratory of Karunya University. A push-pull jack was set up in the structural laboratory. Both the column ends were provided with hinged boundary conditions. At one of the column ends the axial load was applied by using hydraulic jack of 500 kN capacity which has a load measuring arrangement fitted to it.

A transverse load was applied at the free end of the beam by using a push pull jack. A deflectometer was placed on the other side of the beam which shows the deflection that occurs at the point of application of load on the beam. The testing involves pushing of the beam using the push pull jack by applying the load in the pushing direction up to control deflection of 75mm.

Then the pulling load was applied until the beam comes back to its original position. So, one cycle of load reversal was applied to the test specimens. i.e. the beam was pushed from the normal position, then pulled to the normal position, then it was pulled back from the normal position and again pushed back towards the normal position.

The deflectometer readings were noted down at particular load intervals and the deflection of the beam was determined. Typical view of test setup is shown in figure.

II. Discusson of Test Results

a) Results of the Experimental Investigation on Controlled Specimens

BCJ 1 : This specimen has been designed and detailed as per code IS 456:2000. An axial load of 15 % of the safe load on column was applied. The value of the axial load applied was 90KN. The lateral load applied on the beam was at an interval of 5KN. First crack appeared on the tension side of the beam at a load of 19KN. Further three to four cracks developed on the tension side were observed. At a load of 23.5KN first crack developed on the compression side of the beam and further cracks were widen on both compression and tension side of beam. At a load of 38.1KN crack on the tension side started propagating into the column. Spalling of concrete was also started on the compression side of the beam. The application of load was stopped when the controlled deflection of 70.5mm. the load corresponding to that deflection was 47.8KN.

While pulling more cracks occurred at compression side of beam and it propagated into column. Cracks widened and spalling of concrete also observed.

BCJ 2 : This specimen has been designed and detailed as per code IS 456:2000. An axial load of 30 % of the safe load on column was applied. The value of the axial load applied was 180 kN. The lateral load applied on the beam was at an interval of 5KN. First crack appeared on the tension side of the beam at a load of 20.9KN. Further three to four cracks developed on the tension side were observed. At a load 25.9KN first crack developed on the compression side of the beam and further cracks were widen on both compression and tension side of beam. At a load of 41.7KN crack on the tension side started propagating into the column. Spalling of concrete were also started on the compression side of the beam. The application of load was stopped when the controlled deflection of 70.5mm. the load corresponding to that deflection was 52.6KN. While pulling more cracks occurred at compression side of beam and it propagated into column. Cracks widened and spalling of concrete also observed.

BCJ 3 : This specimen has been designed and detailed as per code IS 456:2000. An axial load of 45% of the safe load on column was applied. The value of the axial load applied was 270KN. The lateral load applied on the beam was at an interval of 5KN. First crack appeared on the tension side of the beam at a load of 17.6KN. Further three to four cracks developed on the tension side were observed. At a load of 21.7KN first crack developed on the compression side of the beam and further cracks were widen on both compression and tension side of beam. At a load of 35.2KN crack on the tension side started propagating into the column. Spalling of concrete were also started on the compression side of the beam. The application of load was stopped when the controlled deflection of 70.50 mm. the load corresponding to that deflection was 44.6KN While pulling more cracks occurred at compression side of beam and it propagated into column. Cracks widened and spalling of concrete also observed.

BCJ 4 : This specimen has been designed and detailed as per code IS 13920:1993. An axial load of 15% of the safe load on column was applied. The value of the axial load applied was 90 kN. The lateral load applied on the beam was at an interval of 5KN. First crack appeared on the tension side of the beam at a load of 21.9KN. Further three to four cracks developed on the tension side were observed. At a load of 27KN first crack developed on the compression side of the beam and further cracks were widen on both compression and tension side of beam. At a load of 43.6KN crack on the tension side started propagating into the column. Spalling of concrete were also started on the compression side of the beam. The application of load was stopped when the controlled deflection of 70.5mm. the load corresponding to that deflection was 55KN. While pulling more cracks occurred at compression side of beam and it propagated into column. Cracks widened and spalling of concrete also observed.

BCJ 5 : This specimen has been designed and detailed as per code IS 13920:1993. An axial load of 30% of the safe load on column was applied. The value of the axial load applied was 180KN. The lateral load applied on the beam was at an interval of 5KN. First crack appeared on the tension side of the beam at a load of 24KN. Further three to four cracks developed on the tension side were observed. At a load of 29.8KN first crack developed on the compression side of the beam and further cracks were widen on both compression and tension side of beam. At a load of 48.2KN crack on the tension side started propagating into the column. Spalling of concrete were also started on the compression side of the beam. The application of load was stopped when the controlled deflection of 70.5mm the load corresponding to that deflection was 61KN. While pulling more cracks occurred at compression side of beam and it propagated into column. Cracks widened and spalling of concrete also observed.

BCJ 6 : This specimen has been designed and detailed as per code IS 13920:1993. An axial load of 45% of the safe load on column was applied. The value of the axial load applied was 270KN. The lateral load applied on the beam was at an interval of 5KN. First crack appeared on the tension side of the beam at a load of 20.2KN. Further three to four cracks developed on the tension side were observed. At a load of 25KN first crack developed on the compression side of the beam and further cracks were widen on both compression and tension side of beam. At a load of 40.5KN crack on the tension side started propagating into the column. Spalling of concrete were also started on the compression side of the beam. The application of load was stopped when the controlled deflection of 70.5mm. the load corresponding to that deflection was 51.3KN. While pulling more cracks occurred at compression side of beam and it propagated into column. Cracks widened and spalling of concrete also observed.

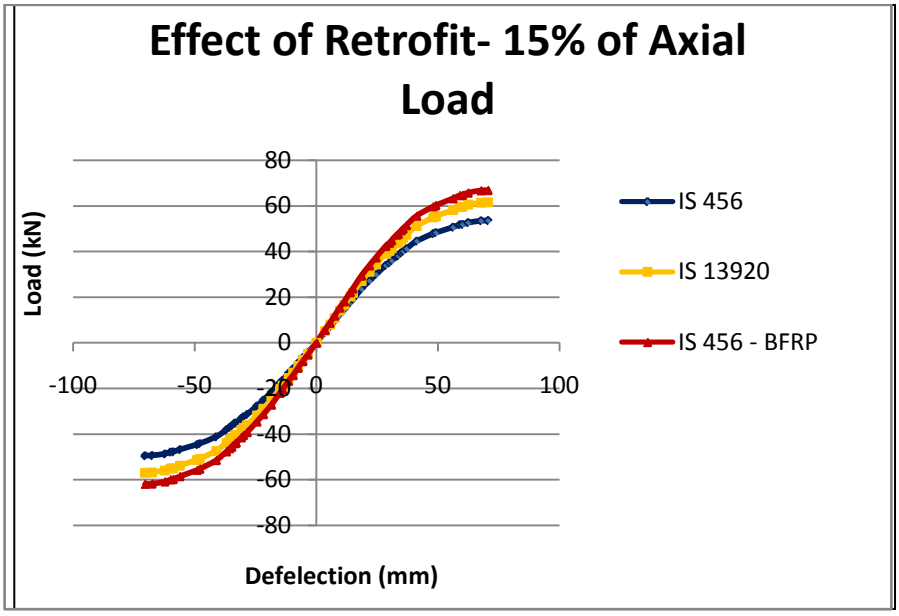
b) Results of the Experimental Investigation on Retrofitted Specimens

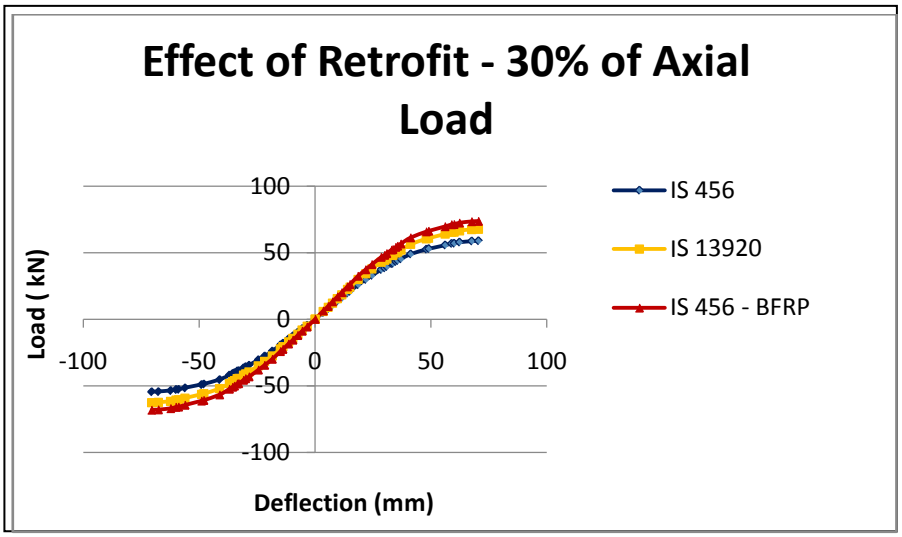
BCJ R1 : This specimen has been retrofitted with Basalt FRP sheets. An axial load of 15% of the safe load on column was applied. The value of the axial load applied was 90KN. The lateral load applied on the beam was at an interval of 5KN. First crack appeared on the tension side of the beam at a load of 23.8KN. Further three to four cracks developed on the tension side were observed. At a load of 29.4KN first crack developed on the compression side of the beam and further cracks were widen on both compression and tension side of

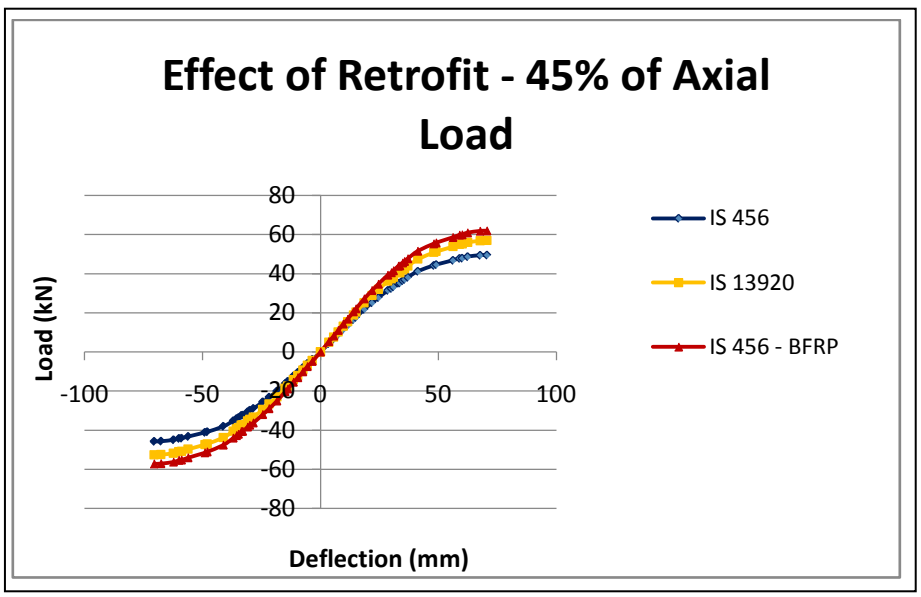
beam. At a load of 47.6KN crack on the tension side started propagating into the column. Spalling of concrete were also started on the compression side of the beam. The application of load was stopped when the controlled deflection of 70.5mm. the load corresponding to that deflection was 59.8KN. While pulling more cracks occurred at compression side of beam and it propagated into column. Cracks widened and spalling of concrete also observed.

BCJ R2 : This specimen has been retrofitted with Basalt FRP sheets. An axial load of 30% of the safe load on column was applied. The value of the axial load applied was 180KN. The lateral load applied on the beam was at an interval of 5KN. First crack appeared on the tension side of the beam at a load of 26.1KN. Further three to four cracks developed on the tension side were observed. At a load of 32.4KN first crack developed on the compression side of the beam and further cracks were widen on both compression and tension side of beam. At a load of 52.4KN crack on the tension side started propagating into the column. Spalling of concrete were also started on the compression side of the beam. The application of load was stopped when the controlled deflection of 70.5mm. the load corresponding to that deflection was 66.3KN. While pulling more cracks occurred at compression side of beam and it propagated into column. Cracks widened and spalling of concrete also observed.

BCJ R3 : This specimen has been retrofitted with Basalt sheets. An axial load of 45% of the safe load on column was applied. The value of the axial load applied was 270KN. The lateral load applied on the beam was at an interval of 5KN. First crack appeared on the tension side of the beam at a load of 22KN. Further three to four cracks developed on the tension side were observed. At a load of 27.1KN first crack developed on the compression side of the beam and further cracks were widen on both compression and tension side of beam. At a load of 44KN crack on the tension side started propagating into the column. Spalling of concrete were also started on the compression side of the beam. The application of load was stopped when the controlled deflection of 70.5mm. the load corresponding to that deflection was 55.3KN. While pulling more cracks occurred at compression side of beam and it propagated into column. Cracks widened and spalling of concrete also observed.







Axial Load on Column (Controlled and Retrofitted Specimens)

III. Finite element Analysis using Ansys

a) Solid 65 (3D Reinforced Concrete Solid) and 45 (3D Structural Solid)

SOLID 65 elements were used to model reinforced concrete problems or reinforced composite materials (FRP). This element has eight nodes each node having three translational degrees of freedom in the nodal X, Y & Z directions as shown in Figure 5.5. The Solid 65 may be used to analyse cracking in tension and crushing in compression and solid 45 element has stress stiffening, large deflection, placticity, large strain capabilities, creep etc. The element may be used to analyse cracking in tension and crushing in compressionUp to three rebar specifications may be defined. The typical solid 65 element was shown in fig.

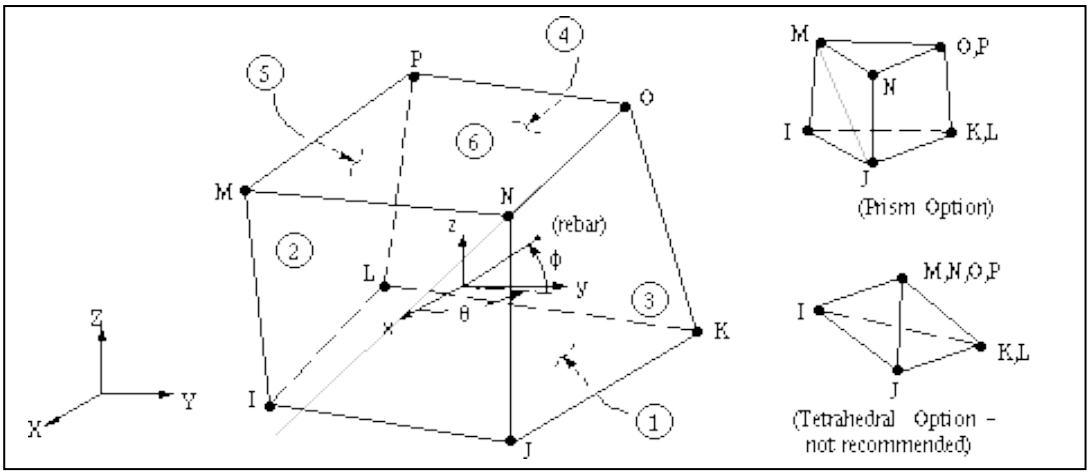
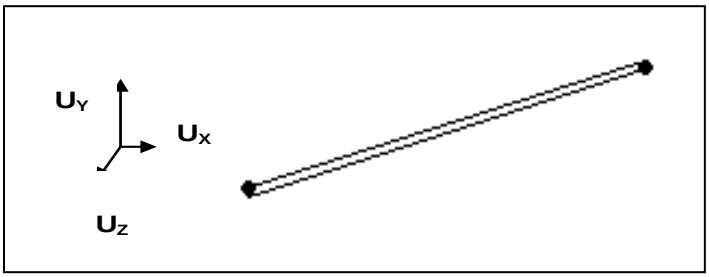


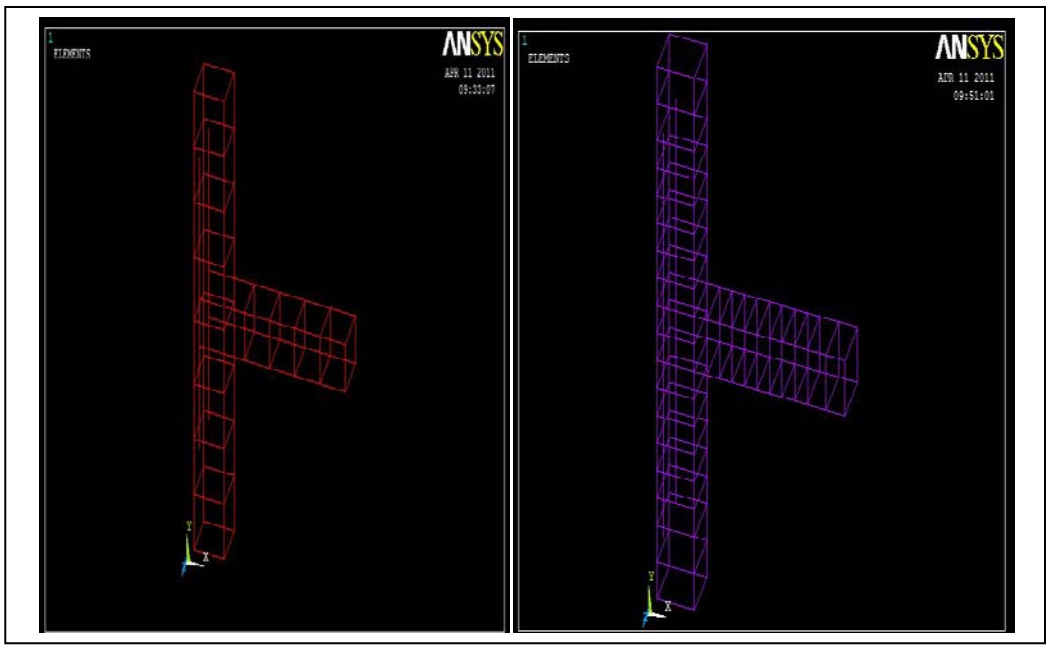
Figure : Solad 65 Element

LINK 8

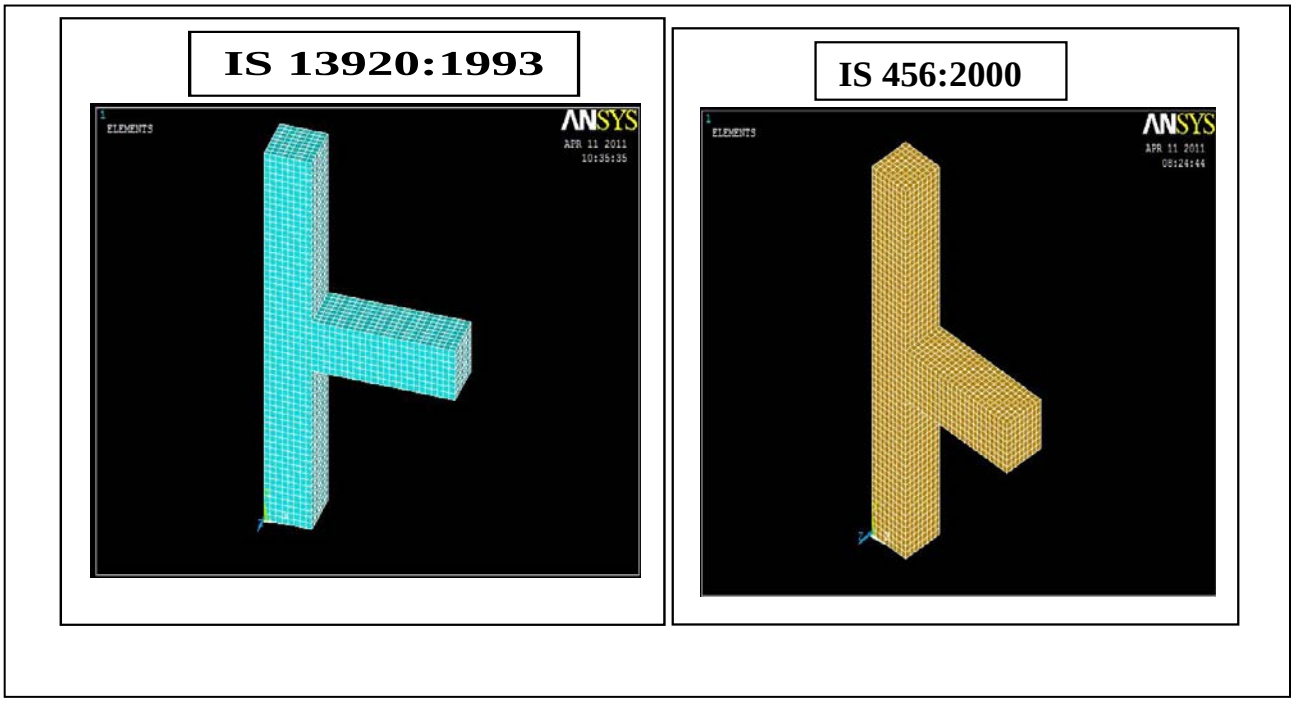
Link 8 Is A Spar Element, Which May Be Used In Variety Of Engineering Applications. Depending Upon The Applications, The Element May Be Thought As A Truss Element, Cable Element, Reinforcing Bar And Bolt. The Three-Dimensional Spar Element Is Having Two Nodes And Each Node Having Three Translational Degrees Of Freedom. This Element Is Capable Of Plasticity, Creep, Swelling And Stress Stiffening Effects.



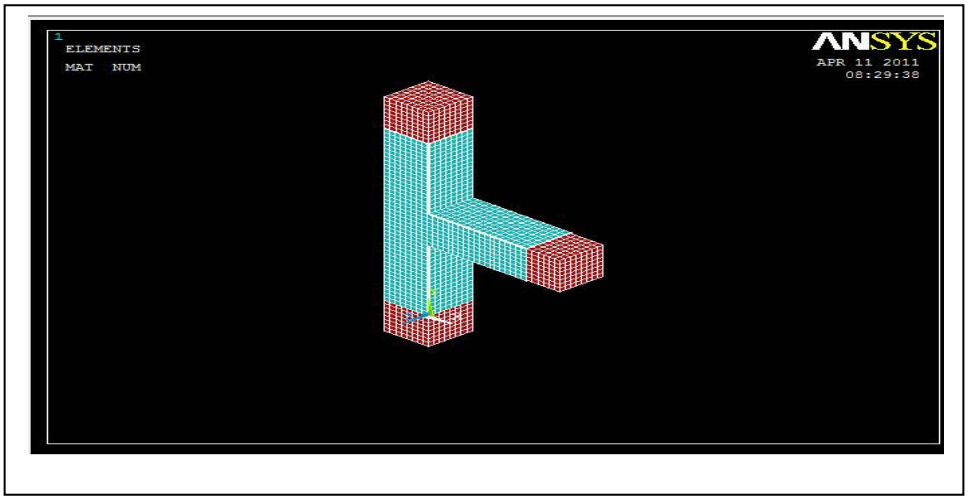
LINK 8 Elements



Reinforcing Detailing on Beam-Column Joints



Fully Modelled Beam-Column Joints

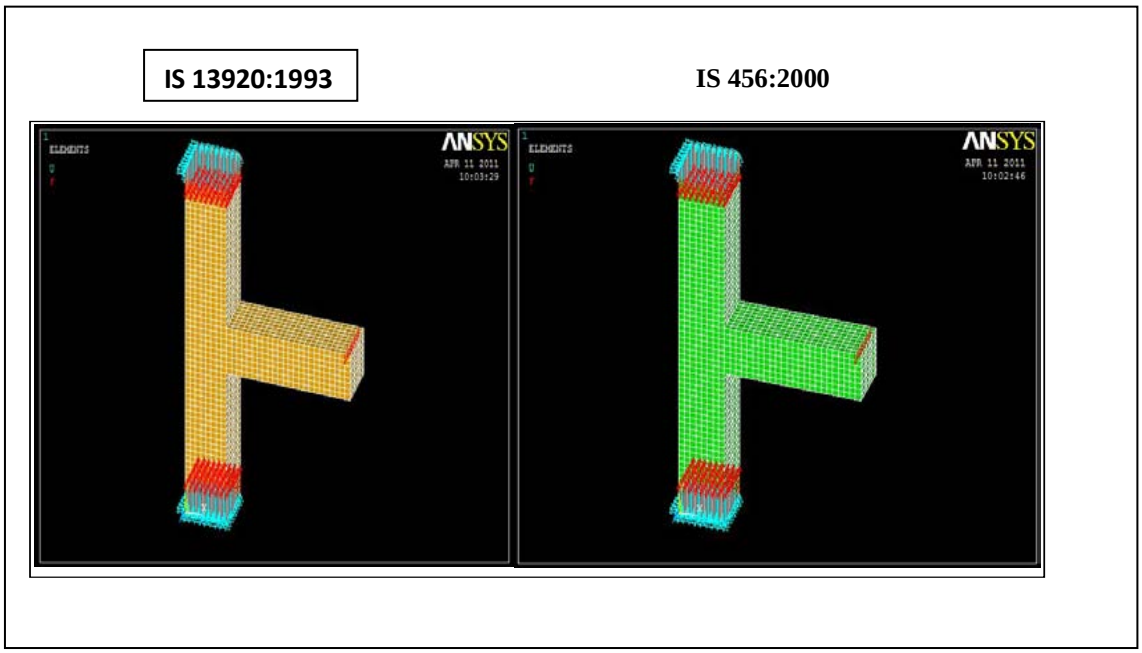


b) Application of Loads and Boundary Condition

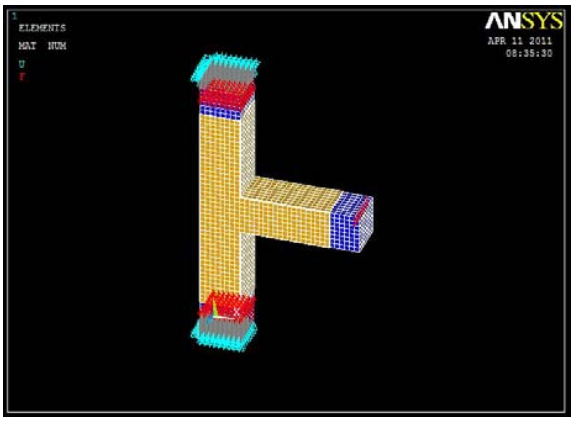
Displacement boundary conditions are needed to constrain the model to get a unique solution. To ensure that the model acts the same way as the experimental beam, boundary conditions need to be applied at points of symmetry, and where the supports and loadings exist. Both ends of the column were provided hinged boundary condition. A lateral load was applied at the free end of the beam. The load applied in model which had detail as per code IS 456:2000 was 23 kN. Similarly in model which had details as per code 13920:1993 ,the load applied was 26 kN. The comparative result were given in table.

AXIAL LOAD	DETAILED AS PER CODE IS 456:2000			DETAILED AS PER CODE IS 13920:1993			IS 456:2000(Retrofitted Specimen)		
	LOAD IN kN	DEFLECTION BY EXPERIMENTAL INVESTIVESTIGATION IN mm	DEFLECTION BY ANSYS RESULT IN mm	LOAD IN kN	DEFLECTION BY EXPERIMENTAL INVESTIVESTIGATION IN mm	DEFLECTION BY ANSYS RESULT IN mm	LOAD IN kN	DEFLECTION BY EXPERIMENTAL INVESTIVE STIGATION IN mm	DEFLECTION BY ANSYS RESULT IN mm
15%	53.5	70.5	52.27	61.5	70.5	59.87	66.9	70.5	51.5
30%	58.9	70.5	52.27	67.7	70.5	56.6	73.6	70.5	48.9
45%	49.5	70.5	56.9	43.8	70.5	47.6	61.9	70.5	49.47

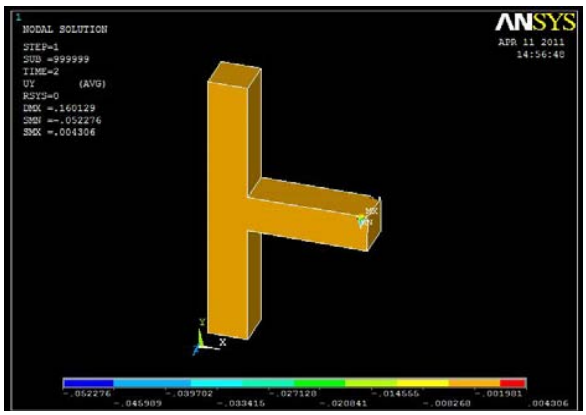
Comparison of Results Between Ansys and Experimental Result



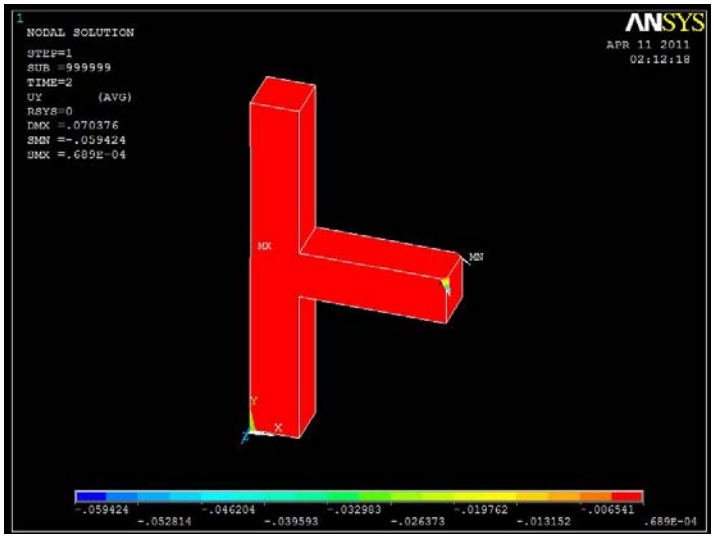
Loading arrangements and boundary condition of Beam-Column Joints



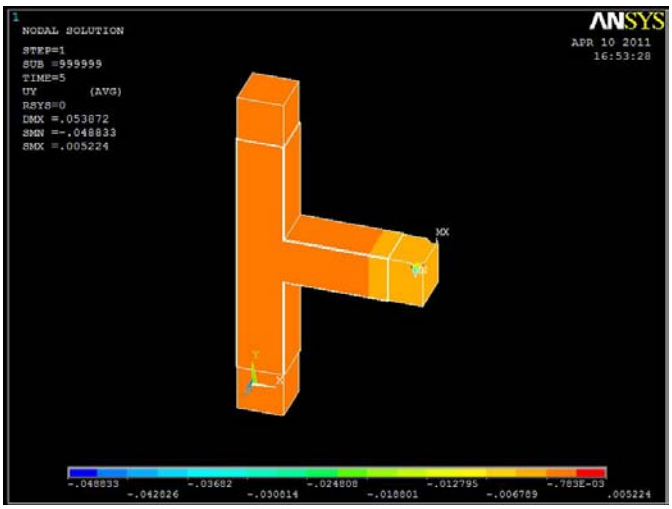
Loading arrangement and boundary condition of Beam-Column Joints retrofitted specimen



Displacement Solution For Beam-Column Joints As Per Code Is 456: 2000 For The Load of 49.5 kN



Displacement Solution For Beam-Column Joints As Per Code IS 13920: 1993 For The Load of 61.5 kN



Displacement Solution For Beam-Column Joints As Per Code IS 456 Retrofitted Specimen 66.9 kN

IV. Conclusions

- In the case of specimens having reinforcement details as per code IS 456:2000, there is an increase of 14.4% in load carrying capacity and 18.87% in energy absorption capacity, when the axial load on column was increased from 15% to 30%.
- In the case of specimens having reinforcement details as per code IS 456:2000, there is an increase of 12.90% in load carrying capacity and 16.61% in energy absorption capacity, when the axial load on column was increased from 15% to 45%.
- In the case of specimens having reinforcement details as per code IS 13920:1993, there is an increase of 16.71% in load carrying capacity and 21.06% in energy absorption capacity, when the axial load on column was increased from 15% to 30%.
- In the case of specimens having reinforcement details as per code IS 13920:1993, there is an increase of 12.25% in load carrying capacity and 14.10% in energy absorption capacity, when the axial load on column was increased from 15% to 45%.
- In the case of specimens retrofitted by Basalt FRP wrapping, there is an increase of 31.89% in load carrying capacity and 33.07% in energy absorption capacity, when the axial load on column was increased from 15% to 30%.

- In the case of specimens retrofitted by Basalt FRP wrapping, there is an increase of 14.58% in load carrying capacity and 16.31% in energy absorption capacity, when the axial load was increased by 15% to 45%.
- In the case of specimens having reinforcement details as per code IS 13920:1993 with 15% of axial loading on the column, there was an increase of 18.5% in load carrying capacity and 19.5% increase in energy absorption capacity than the specimens with reinforcement details as per code IS 456:2000 with same axial load on column.
- In the case of specimens having reinforcement details as per code IS 13920:1993 with 30% of axial loading on the column, there was an increase of 17.4% in load carrying capacity and 18.4% increase in energy absorption capacity than the specimens with reinforcement details as per code IS 456:2000 with same axial load on column..
- In the case of specimens having reinforcement details as per code IS 13920:1993 with 45% of axial loading on the column, there was an increase of 16.3% in load carrying capacity and 17.3% increase in energy absorption capacity than the specimens with reinforcement details as per code IS 456:2000 with same axial load on column.
- In the case of specimens having reinforcement detailing as per code IS 456:2000 with 15% of axial load on column, retrofitted with–Basalt FRP wrapping, there was an increase of 32.6% in load carrying capacity and 29.5% increase in energy absorption capacity than the specimens with reinforcement detailing as per code IS 456:2000 with same axial loading on column.
- In the case of specimens having reinforcement detailing as per code IS 456:2000 with 30% of axial load on column, retrofitted with Basalt FRP wrapping, there was an increase of 35.3% in load carrying capacity and 31.5% increase in energy absorption capacity than the specimens with reinforcement detailing as per code IS 456:2000 with same axial loading on column.
- In the case of specimens having reinforcement detailing as per code IS 456:2000 with 45% of axial load on column, retrofitted with Basalt FRP wrapping, there was an increase of 33.91% in load carrying capacity and 34.84% increase in energy absorption capacity than the specimens with reinforcement detailing as per code IS 456:2000 with same axial loading on column.
- Experimental test result of IS 456-2000 specimen when compared with ANSYS result was found to be less with error of 34.87%.
- Experimental test result of IS13920-1993 specimen when compared with ANSYS result was found to be with less of 34.87%.
- Experimental test result of IS 456-2000 retrofitted specimen when compared with ANSYS result was found to be less with error of 36.89%.

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Long Term Performance Test of Low Span Low Cost Masonry Slab (Without Reinforcement) Under Static Load, Repeated Load and Impact

By M. Jobaer Hasan, Mahzabin Afroz & M. M. Hossain
Khulna University of Engineering & Technology, Bangladesh

Abstract - In residential buildings, low cost is a vital demand. Slab made of brick module with or without beam is found to be practiced locally. Moreover brick masonry slab is easy to construct and durable with respect to fire proofing and corrosion if nominal or zero reinforcement is possible. Therefore a study was under taken in the Department of Civil Engineering, KUET to investigate the long-term performance of brick masonry slab of dimensions 1.52m × 3.65m, with a slab thickness of 75mm. Tests were performed subjected to static, repeated and impact loading system. Test results revealed that brick masonry slab did not failed and no crack were observed though it was loaded by a uniform pressure of 12kN/m2. Similar phenomena were observed when repeated load up to12kN/m2 was imposed on the slab. However punching shear failure was observed when an impact load was applied 9 times by a hammer of 23kg of 1m free fall. Combined failure both in joint and brick module was observed.

Keywords : *masonry slab, full scale test, long term effect, static load, repeated load and impact load.*

GJRE-E Classification : *FOR Code: 290801, 090599*

Strictly as per the compliance and regulations of :

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I. Introduction

The construction of using stone, brick, block etc is termed as masonry. It may be defined as building units bonded together with mortar. The rapid progress over recent past in the understanding of the materials and considerable advances in the method of design have increased acceptance of load bearing masonry as a variable structural material. Brick masonry is one of the oldest building materials comparatively superior to other alternatives in terms of appearance, durability and cost (Hossain M. M. et al., 1997). Roof system of a residential building is an indispensable part. There are several type of roof system which are usually constructed in rural and urban areas namely, conventional R.C.C. slab beam, wooden rafter and beam covered with tile followed by lime surki mortar finish, brick masonry roof reinforced by MS bar or other indigenous material. Sometime unreinforced brick masonry is found to be constructed from long past. Effort of lowering cost has become burning need for low income group of people. Room with comparatively short span length is used in rural adobe buildings. For cost optimization and broaden utility, its possibility needs to be verified by full scale tests.

II. Background of Research

Reinforced brick slab are widely used in low cost rural housing. Design and code related to reinforced brick slab are well established (Dayaratnam P, 1988 and Kumar S, 2005). Higher rate of corrosion in reinforcing steel and high cost of reinforcement has necessitated the study on brick slab without reinforcement for the interest of economy and durability of the slab (Siddiqi and Ashraf, 2000). Rabbani and Nahid (2006) investigated the parametric study on more than 30 brick slabs without reinforcement. Parameter included – brick line, span and filler. Figure 1 shows one of their typical laying pattern and Figure 2 shows the loading arrangement for the test of slab.

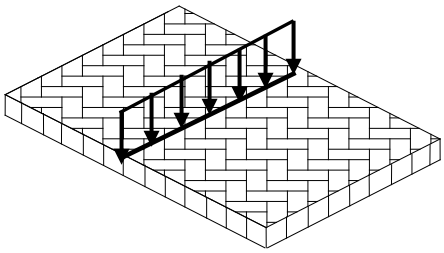

Figure 1 : Model of herring bone bond slab of size (0.91m×0.61m×0.075m)

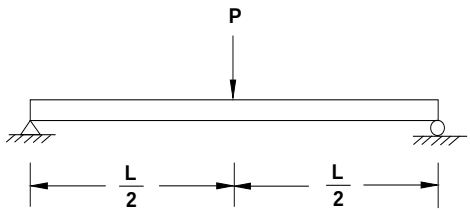

Figure 2 : Loading arrangement of model slab (L=787 mm)

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They concluded that herring bone bond masonry slab of 75 mm thickness can resist flexural stress of about 250 psi. Therefore in this study low cost housing masonry slab of 3.65m×1.52m ×0.075m has been constructed and tested with uniform distributed load, repeated load and impact.

III. Preparation of Test Slab

In this study, a two panel masonry slab each of 3m×1.5m ×0.075m are cast with brick module placed flat providing 0.075m thickness for the slab. The interspaces between the modules (12.7 mm) are sealed with mortar.

a) Materials Specifications

First class brick the average compressive strength 30MPa.

Cement mortar ratio 1:1.5

Ordinary Portland Cement

Washed Local sand with fineness modulus of 1.5

b) Construction Sequences

First of all, wooden platform was prepared and leveled before laying the bricks. Bricks are then laid in staggering pattern placed with frog mark at to side keeping 12.7mm. Layout and support position of the masonry slab has shown in Figure 3. On the other hand, Figure 4 and 5 shows the detailing of the support size in cross-section and long section respectively. A 75mm thick slab was made keeping 12.7mm gap in between two adjacent bricks. Figure 6 shows a close view photograph of the same. Top surface of the slab was finished with 12.7 mm mortar with neat finish. After 24 hours a 75 mm height of brick border was made to store water for curing purposes. After completing 28 days of curing period the formwork was removed and the slab was prepared for test.

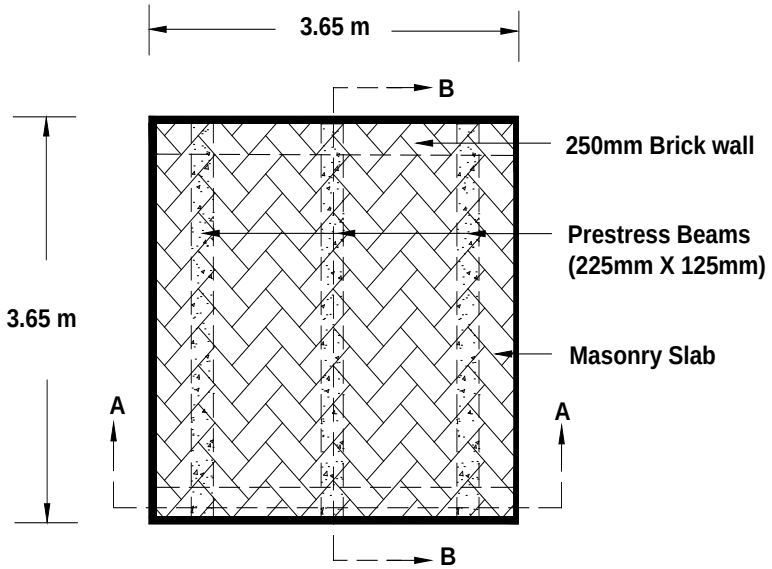

Figure 3 : Layout and support position of slab

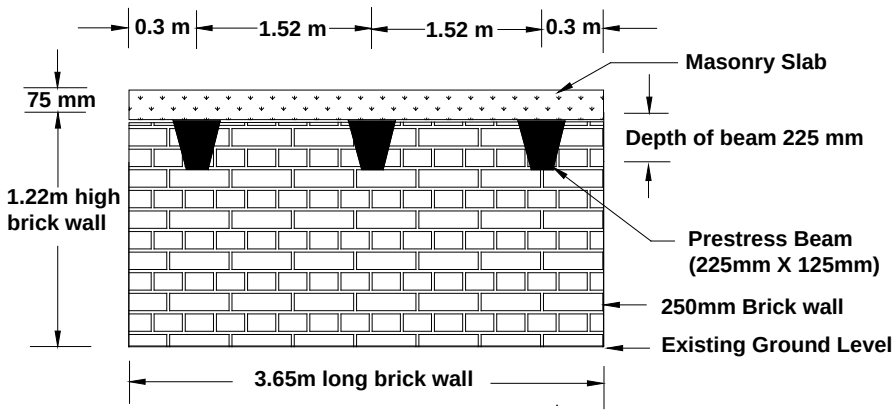

Figure 4 : Section A-A

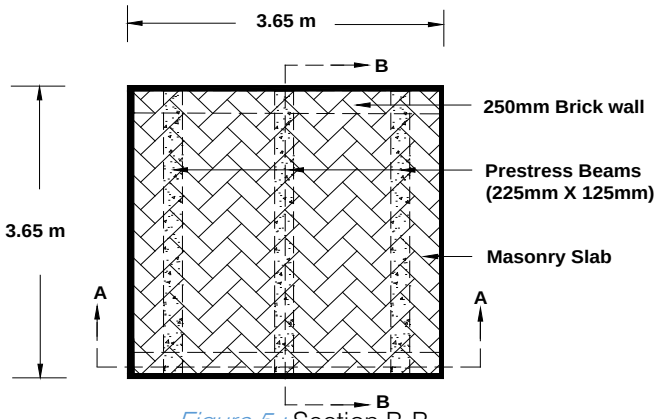

Figure 5 : Section B-B

Figure 6 : Photograph of Close view - showing the interspaces 12.7mm in bricks

c) Instrumentation and Testing

Instrumentation and testing was performed in two phase. In first phase, only load bearing capacity of the full scale slab was tested and the test was done after 28 days of slab construction. Second phase test was done after 5 years of slab construction. This paper deals with the instrumentation and results of the second phase.

Testing of second phase involved the application of static load, repeated load and impact load.

To perform the static load test, a brick wall of height 1.2m and 125mm in thickness was constructed around the 3.65m×1.52m slab. Then water pump was used to fill the 3.65m×1.52m×1.2m chamber on the slab. Linear Voltage Displacement Transducers (LVDTs), portable data logger and computer arrangements were used for data acquisition. LVDTs were instrumented as shown in Figure 8 and connected with data logger (Figure 9).

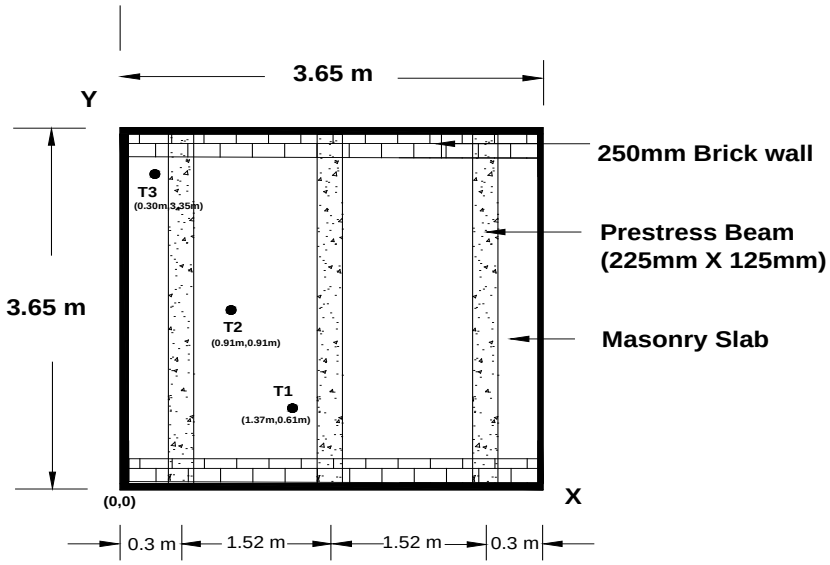

Figure 7 : Location of LVDTs

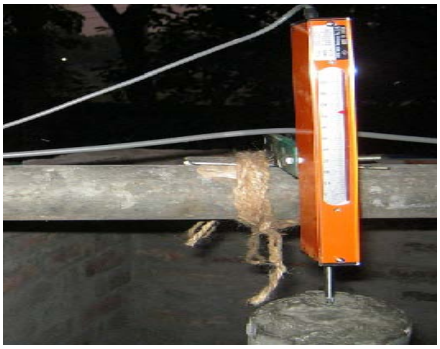

Figure 8 : LVDT Setup

Figure 9 : Portable Data Logger

Figure 10 : Data Acquisition Devices

To perform the repeated load test, similar instrum entation was done. In this case, the height of water was increased again deceased gradually with respect to time and the reading changes in the data acquisition devices were observed. This was repeated 10 times.

To perform the impact load test on the masonry slab a weight of 23 kg was set to free fall on the slab from a height of 1 m as shown in Figure 11. Figure 12 shows the indigenous arrangement for the application of impact load.

Figure 11 : Impact test setup with round hammer ball, 1m free fall on slab

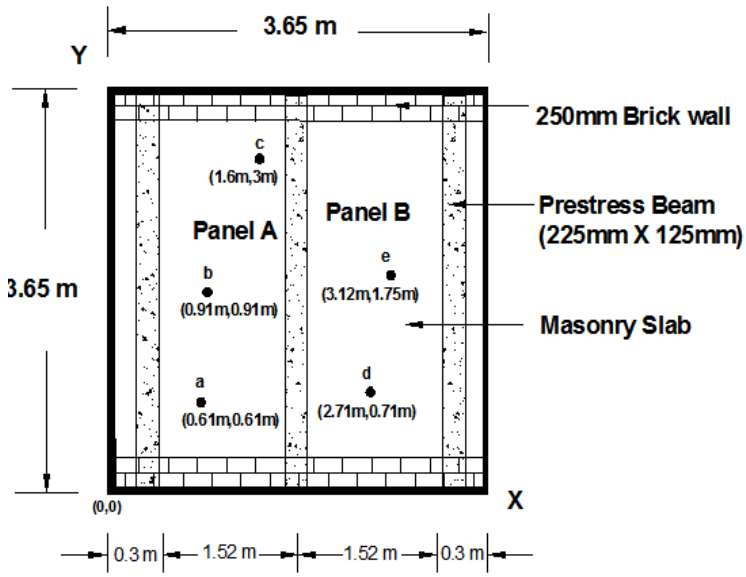

Figure 12 : Location on slab where impact loadwas applied

IV. Test Results and Discussion

a) Final Phase

This has been done after the construction period of 5 years and data acquisition systems corresponding to deformation such as LVDT's and strain gauges in slab has been taken. In this phase mainly three types of loading were induced on s
ab panels, namely:

a. Static load
b. Repeated load and
c. Impact load

i. *Static loading on slab panel*

From the test no significant change in deformation was recorded from the data acquisition devices. However the slab carried a water column height of 1.22m on the area of 3.65m×1.52m which equivalent to 12kN/m2. Hence the slab carried a uniform distributed load 4 times than traditional load of residential buildings. Moreover no crack and leakage of slab panel was observed.

ii. *Repeated loading on slab panel*

No significant change in deformation was observed when repeated was induced on slab panel.

iii. *Impact loading on slab panels*

In this case impact hammer was dropped to five different locations as shown in 13 on the slab.

Table 2 : Result of Impact Test

Location		Number of drop applied to fail	Equivalent Diameter of the Punched section (mm)
Panel A	a	7	40
	b	9	30
	c	8	35
Panel B	d	8	50
	e	6	55

Table 2 shows the number of drop required for punching failure. From the test it was observed that the masonry brick slab though a brittle material, it did not failed catastrophically rather than just failed locally due to punching. In Panel A at 'b' point was tested first, but no significant crack was showed after punch of this point. On the other hand when 'c' point was tested it showed few cracks as shown in Figure 13. However significant cracks were observed when impact load were induced at points'd' and 'e' of Panel B (Figure 14 and 15). Crack patterns showed the brick failure of the slab rather than joint failure. Hence it reveals combined action of the matrices while the structure induced to load.

Figure 13 : Crack at Point c

Figure 14 : Crack at Point e

Figure 15 : Crack at Point d

Maximum flexural stress induced in the slab while applying the impact load can be calculated from equation,

$$\sigma = \sqrt{\frac{6mghEc^2}{LI}}, \text{(Pytel A. and Singer L. F.,1999)}$$

$$Where, m = 23kg$$

$$g = 9.81m/s^2$$

$$h = 1m$$

$$E = 670MPa, (RosenhauptS.,1962)$$

$$c = 37.5mm = 0.0375m$$

$$L = 1.52m$$

$$I = \frac{bh^3}{12} = \frac{3.15 \times 0.075^3}{12} = 1.107 \times 10^{-04} m^4$$

Therefore maximum flexural stress developed in the masonry slab while impact load induced on it.

$$\sigma = 2753259.28N/m^2 = 2.75MPa$$

V. Conclusions

In this study low span full-scale masonry slab without reinforcement has been investigated. Following conclusions can be made from this investigation:

- Masonry slab (3.65m×1.52m×0.075m) without reinforcement carried uniform distributed load of 4 times than conventional residential building after its construction period of 5 years. No leakage of water ensured absence of cracks in the slab panels.
- Slab carried repeated load 10 times while varying the height of water pressure. However no cracks and no change in deformation were found.
- Slab carried impact load of 23kg hammer ball from 1m height at least 6 times to maximum 9 times before failure and punching of slab were observed with diameter 5cm.
- Flexural stress of masonry slab under impact load was calculated as 2.74 MPa.

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Reasons and Ways to Redefine Seismic Intensity Relying on Instrumental Information

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Abstract - The shortcomings of the "traditional" concept of seismic intensity from the viewpoint of requirements of accuracy of input data to be used in specific engineering activities are recognized on one hand. An illustrative case of deriving wrong conclusions due to some of these shortcomings is referred to. On the other hand, the importance of the concept of seismic intensity for the management of a large, worldwide, treasury of information and for some current activities too, is also recognized. An attempt of bridging the gap between engineering requirements and the use of the concept of seismic intensity is presented, introducing alternative approaches to the definition of seismic intensity, relying on specific instrumental information. The main reasons of proposals are discussed. The main starting points are presented too. This is followed by analytical developments related to the features of alternative definitions proposed. Some illustrative cases dealt with on the basis of these developments are then presented. A short look at conclusions derived and on desirable future activities is then dealt with.

Keywords : seismic intensity, global intensity, spectrum based intensity, intensity based on arias type integral, intensity based on fourier spectrum, frequency related intensity, averaged intensity, intensity spectrum.

GJRE-E Classification : FOR Code: 299904, 090599

Strictly as per the compliance and regulations of :

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Horea Sandi

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I. Introduction

The concept of seismic intensity, aimed as a first historical attempt to quantify the severity of ground motion during earthquakes, has played an important role in the development of seismology and is still widely used by seismologists. The main functions of this concept may be stated to be:

- Evaluation of the severity of actual ground motions for which appropriate post-earthquake surveys are available (basically, rather recent events),
- Evaluation of the severity of ground motions for which information at hand is scarce (usually, events of the more remote past, "historical earthquakes" included),
- characterization of the reference severity of local seismic conditions in order to specify criteria of earthquake protection for a definite area.

In case one takes as a reference the two most recently endorsed European seismic intensity scales, namely MSK-76 [Medvedev, 1977] and its successor EMS-98 [Grünthal, 1998], it turns out that seismic intensity is quantified in scalar, discrete, terms. This way of quantification provides scarce information and is by far not satisfactory as a tool for specification of data required at present for engineering activities specific to earthquake protection. This fact led practically to a rejection of seismic intensity as a tool for current engineering practice. On the other hand, seismic intensity represents an often unique tool available for quantifying ground motion severity, especially in case of absence of instrumental information, and this happened for all earthquakes of the more remote past and quite frequently even for recent events. This is why the concept of seismic intensity should be not rejected, but rather adapted, made compatible, with up to date engineering know how.

Following developments represent an attempt to contribute to this task. They rely on the quite longtime concern of the author, on cooperation for case studies with colleagues mentioned in the acknowledgements and most recently, on the international cooperation in the frame of the Project "Quantification of Earthquake Action of Structures" (2005 – 2008). This latter project [Sandi et al., 2010a] benefited from support provided by the NATO Office in Brussels, in the frame of the program "Science for Peace".

II. Main Reasons of Proposals

Current knowledge in the field of structural dynamics makes it possible to predetermine by means of engineering analysis the features of effects of a given, well specified, ground motion upon a well characterized structure. The significance of spectral contents and of possible directionality of ground motion is made clear in this sense. On the other hand, looking at the MSK and EMS scales referred to, some significant features revealing their limits and shortcomings can be mentioned. Both scales are based on the use of macroseismic criteria, implicitly postulated according to the philosophy on which these scales rely, to be the most relevant ones. Macroseismic criteria are carefully specified, especially in the frame of the EMS scale. The MSK scale presents in an annex also some instrumental criteria, referring to *PGA* (peak ground acceleration), *PGV* (peak ground velocity) and peak displacement of a standard pendulum (Medvedev's "*SBM*" pendulum, having a natural period of 0.25 s and a logarithmic decrement of 0.5). The criteria postulated are consistent with a standard type of acceleration response spectrum, as adopted in [Medvedev, 1962]. This has a standard

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velocity / acceleration corner period of 0.5 s, a constant value for $T \leq 0.5$ s and values proportional to $1/T$ for $T > 0.5$ s. These latter criteria are assumed to be of secondary importance. The EMS scale presents no instrumental criteria, in spite of explicitly recognizing, in the comments to the scale, that a complete, correct, record fully characterizes local ground motion. It turns thus out that the criteria of the MSK and EMS scales are blind towards the spectral and directional features of ground motion, which in fact so strongly influence the destructive potential of ground motion upon various categories of elements at risk. This blindness may have heavy consequences.

A case study in this sense was presented in [Sandi & Borcia, 2010b]. It was shown how neglecting the features of spectral contents of ground motion led in Romania in the past to erroneous seismic zonation, which could be corrected only after making clear the conclusions derived on the basis of quite rich instrumental information obtained during the strong earthquakes of 1977.03.04, 1986.08.30, 1990.05.30 and 1990.05.31. The initial interpretation (according to MCS and MSK scales respectively) of macroseismic information obtained during the destructive earthquakes of 1940.11.10 and 1977.03.04 led to a zonation map according to which the City of Bucharest was located in a local island of intensity VIII, surrounded by a zone of intensity VII. This happened in spite of the fact that geological conditions were not justifying such a difference. When instrumental information became available, it turned out subsequently to the four events of 1977, 1986 and 1990, that the seismic conditions are quite similar for the City of Bucharest and its surroundings and this led to attributing to city and surroundings both, the same intensity, VIII. Why did the use of macroseismic criteria lead to wrong conclusions? This happened because in case of significantly strong motions the main peak of the response spectrum for absolute accelerations corresponded, inside Bucharest as for its surroundings, to a quite long period, of about 1.5 s. This led to more severe earthquake effects inside the city (where taller buildings exist) than for the surroundings (where the building stock was low rise), ergo to the survey conclusion that intensity would have been higher inside Bucharest than for the surroundings.

III. Fundamentals of Proposals

The proposals presented further on, which are intended to be compatible with the requirements of information specific to engineering activities, rely on the use, as a basic source of information about ground motion, of appropriate accelerograms. Following developments distinguish between *traditional macroseismic criteria*, like those specified by MSK and EMS scales, and *instrumental criteria*, relying on the use of results of appropriate processing of accelerographic data. Recognizing that parameters like *PGA* or *PGV* are of questionable relevance for the destructive potential of ground motion; some alternative starting points were adopted. The main objective of the proposals developed was to find ways to make available some criteria that lead to a best compatibility with macroseismic criteria *when the use of macroseismic criteria leads to results believed to be reasonable*, but also to correct the outcome of use of macroseismic criteria *when the use of the latter ones appears to lead to wrong estimates*. It is of course hard if not impossible to characterize or categorize in rigorous terms the cases in which macroseismic approaches lead to realistic or unrealistic results, but practical experience can compensate for the lack of firm criteria of evaluating the correctness of outcomes of field surveys. This means, of course, specific analyses concerning various practical cases and appropriate expert judgment.

The system proposed, called SAIS, is organized as follows. Three solutions were envisaged in order to adopt appropriate definitions of (global) seismic intensity. A first solution, *spectrum based intensity* (I_S) was to use the characteristics of convex envelope response spectra, like those used in order to specify seismic input for the engineering verification of NPPs [Sandi, 1986]. A second solution (I_A) was to use an *integral of square of acceleration*, as adopted by Arias [Arias, 1970]. A third solution (envisaged by Arias too, I_F) was to use *integrals of absolute squares of Fourier spectra of acceleration*. Note that the latter two solutions (introduced in [Sandi & Floricel, 1998]) can be generalized (in case one considers also products of acceleration time histories possibly corresponding to different directions under the integral) in order to define intensity tensors which would make it possible at their turn to explicitly characterize motion directionality etc.

Being aware of the importance of the spectral content of ground motion, the consideration of just global intensities was considered insufficient. So, frequency dependent intensities were considered too (note that oscillation frequency, quantified in Hz, is denoted further on by φ). Corresponding to I_S, a frequency dependent intensity denoted i_s (φ) was defined on the basis of the product of ordinates of response spectra of absolute acceleration, s_{aa} (φ, ζ), and of absolute velocity, s_{va} (φ, ζ) (both of them for $\zeta =$ 0.05 critical damping) respectively. A *frequency dependent intensity* i_d (φ), homologous to I_A, was defined on the basis of quadratic integrals of acceleration (characterizing at their turn "*motion destructiveness*"), *this time not of ground motion, but of a pendulum having an undamped natural frequency* φ (and a 5% critical damping). A *frequency dependent intensity* i_f (φ), *based on Fourier spectra*, homologous to I_F, was defined on the basis of quadratic integrals of Fourier spectra of acceleration of the same pendulum.

Table 1 : System of Instrumental Criteria for Intensity Assessment

Name	Symbols used for intensities: * global, I_X; ** related to a frequency φ, $i_x(\varphi)$; *** averaged upon an interval (φ', φ''), $i_x^{\sim}(\varphi', \varphi'')$.			Source of definition / comments
	*	**	***	
Spectrum based intensities	I_S	$i_s(\varphi)$	$i_s^{\sim}(\varphi', \varphi'')$	Linear response spectra for absolute accelerations and velocities / use of *EPA*, *EPV*, redefined as *EPAS*, *EPVS* respectively (see relations (2)); averaging rules specified.
Intensities based on Arias' type integral	I_A	$i_d(\varphi)$	$i_d^{\sim}(\varphi', \varphi'')$	Integrals of square of acceleration of ground (for I_A), or of pendulum of natural frequency φ (for $i_d(\varphi)$) / extensible to tensorial definition; averaging rules specified.
Intensities based on quadratic integrals of Fourier images	I_F ($\equiv I_A$)	$i_f(\varphi)$	$i_f^{\sim}(\varphi', \varphi'')$	Integrals of squares of Fourier image of acceleration (for I_F), or absolute squares of Fourier images of a pendulum (for $i_f(\varphi)$) / extensible to tensorial definition; averaging rules specified.

These definitions make it possible to consider Intensity spectra, as functions (in principle continuous) of φ. It was felt that, besides frequency dependent intensities, intensities averaged upon a frequency interval should be defined. Using an averaging rule specified in next section, the *averaged intensities* $i_s^{\sim}(\varphi', \varphi'')$, $i_d^{\sim}(\varphi', \varphi'')$ and $i_f^{\sim}(\varphi', \varphi'')$ respectively were introduced besides the *frequency dependent intensities* $i_s(\varphi)$, $i_d(\varphi)$ and $i_f(\varphi)$, in order to define on this basis also discrete intensity spectra. An overview of the system is given in Table 1.

Note also that the subscript X means any of the subscripts S, A or F, while the subscript x means any of the subscripts s, d or f.

The qualitative definitions presented previously are followed by analytical definitions given in next section.

IV. Analytical Developments

a) Alternative Intensity Definitions

The alternative measures of intensity proposed, pertaining to categories I_X, $i_x(\varphi)$ and $i_x^{\sim}(\varphi', \varphi'')$, are thus defined on the basis of homologous entities Q_X, $q_x(\varphi)$ and $q_x^{\sim}(\varphi', \varphi'')$, having a kinematic sense, defined at their turn subsequently. All quantities Q_X and q_x defined on the basis of instrumental data, which are used in order to estimate intensities, have a physical dimension $L^2 T^{-3}$ and are quantified in terms of $m^2 s^{-3}$. The relations between the two categories of entities are respectively

$$I_X = I_{XQ} + I_{X0} = \log_b Q_X + I_{X0} \quad (1.a)$$

$$i_x(\varphi) = i_{xq}(\varphi) + i_{x0} = \log_b q_x(\varphi) + i_{x0} \quad (1.b)$$

$$i_x^{\sim}(\varphi', \varphi'') = \quad (1.c)$$

$$= i_{xq}^{\sim}(\varphi', \varphi'') + i_{x0} = \log_b q_{xq}^{\sim}(\varphi', \varphi'') + i_{x0}$$

The choice of this way of definitions was suggested first by the instrumental criteria of the MSK scale which adopts, for intensity degrees VI to IX, a geometric progression having a rate of 2.0. This led to a logarithm base $b = 2^2 = 4$. On the other hand, an extensive statistical survey performed by Aptikaev [Aptikaev, 2005] where the relationship between macroseismic intensities and kinematic parameters was investigated, led to the conclusion that geometric progressions for acceleration and velocity amplitudes are quite appropriate in principle, but the corresponding rates are different: they are close to 2.5 for acceleration amplitudes and to 3.0 for velocity amplitudes. This reveals on one hand a tendency of decrease of dominant frequencies with increasing intensities and suggests, on the other hand, a value $b \approx 2.5 \times 3.0 = 7.5$. Since the adoption of a certain logarithm base b represents a significant problem, the implications of a

possible change of it are discussed too towards the end of this subsection.

The definitions of entities Q_X were adopted as follows:

(a) The definition of Q_S was suggested by the concepts of EPA (effective peak acceleration) and EPV (effective peak velocity) introduced by Newmark & Hall [ATC, 1986], which were somewhat modified as.

$$EPAS = \max_{\varphi} s_{aa}(\varphi, 0.05) / 2.5 \quad (2.a)$$

$$EPVS = \max_{\varphi} s_{va}(\varphi, 0.05) / 2.5 \quad (2.b)$$

where $s_{aa}(\varphi, \zeta)$ and $s_{va}(\varphi, \zeta)$ represent the response spectra of absolute acceleration and absolute velocity respectively (quantified for $\zeta = 0.05$ critical damping). On this basis the parameter Q_S was defined as

$$Q_S = EPAS\,(\mathrm{m/s^2}) \times EPVS\,(\mathrm{m/s}) \quad (3)$$

and may be used as a kind of measure of the area underneath a polygonal, convex, corresponding design spectrum (using a log-log scale), characterized by a corner frequency φ_c,

$$\varphi_c = EPAS / (2\pi \times EPVS) \quad (4)$$

(b) The definition of Q_A was based on an Arias type integral,

$$Q_A = \int [w_g(t)]^2 \, dt \quad (5)$$

(the subscript g stands here for "ground") and may be extended to the case of considering ground motion along different (orthogonal) directions i, j

$$Q_{Aij} = \int [w_{gi}(t)\, w_{gj}(t)] \, dt \quad (5')$$

in case one intends to develop an in depth investigation of directional features of ground motion.

(c) The definition of Q_F was based on an integral of the Fourier spectrum of acceleration, $w_g^{(\varphi)}(\varphi)$,

$$Q_F = \int | w_g^{(\varphi)}(\varphi) |^2 \, d\varphi \quad (6)$$

One has

$$w_g^{(\varphi)}(\varphi) = \int_{-\infty}^{\infty} \exp(-2\pi i\, \varphi\, t)\, w_g(t)] \, dt \quad (7a)$$

$$w_g(t) = \int_{-\infty}^{\infty} \exp(2\pi i\, \varphi\, t)\, w_g^{(\varphi)}(\varphi) \, d\varphi \quad (7b)$$

Note that, due to properties of the Fourier transformation, one has

$$Q_A \equiv 2 \times Q_F \quad (8)$$

The definitions of entities $q_x(\varphi)$ were adopted as follows:

(d) The definition of $q_s(\varphi)$ is based on the use of response spectra of absolute accelerations and velocities.,

$$q_s(\varphi) = s_{aa}(\varphi, 0.05) \times s_{va}(\varphi, 0.05) \quad (9)$$

The definition of $q_d(\varphi)$ is based on the use of an Arias type integral, where instead of an integrand consisting of the square of ground motion acceleration $w_g(t)$, as in the definition of Q_A, one should adopt an integrand consisting of the square of acceleration $w_p(t; \varphi, 0.05))$ of the mass of a pendulum (on which ground motion is acting). Thus pendulum has the (undamped) natural frequency φ and a $\zeta = 0.05$ critical damping,

$$q_d(\varphi) = \int [w_p(t; \varphi, 0.05)]^2 \, dt \quad (10)$$

So, a generalization of consideration for the input of the ground motion, as introduced by Arias, occurs (of course, in case $\varphi \to \infty$, the definition becomes directly related to Arias' idea).

(e) The definition of $q_f(\varphi)$ is based on the use of the Fourier image of ground motion acceleration, $w_g^{(\varphi)}(\varphi)$,

$$q_f(\varphi) = \varphi \, | w_g^{(\varphi)}(\varphi) |^2 \quad (11)$$

Obviously, one has

$$Q_F = \int q_f(\varphi) \, d\varphi / \varphi \quad (12)$$

Note also that the definitions {5), (10) and (11) can be extended too to tensorial definitions homologous to (5').

The definitions of entities $q_x^{\sim}(\varphi', \varphi'')$ are based on a common averaging rule,

$$q_x^{\sim}(\varphi', \varphi'') = [1 / \ln(\varphi'' / \varphi'')] \times \int_{\varphi'}^{\varphi''} q_x(\varphi) \, d\varphi / \varphi \quad (13)$$

In case one wants to average the intensities corresponding to two orthogonal (horizontal) directions of ground motion, denoted by indices 1 and 2 respectively, the corresponding rules to be used will be

$$Q_{X12} = (Q_{X1} + Q_{X2}) / 2 \quad (14.a)$$

$$q_{x12}(\varphi) = [q_{x1}(\varphi) + q_{x2}(\varphi)] / 2 \quad (14.b)$$

$$q_{x12}^{\sim}(\varphi', \varphi'') = [q_{x1}^{\sim}(\varphi', \varphi'') + q_{x2}^{\sim}(\varphi', \varphi'')] / 2 \quad (14.c)$$

It is interesting to compare global intensities I_X with some homologous average intensities $i_x^{\sim}(\varphi', \varphi'')$, related to an interval (φ', φ'') assumed to be appropriate for this purpose. It was estimated that the most appropriate averaging interval is (0.25 Hz, 16.0 Hz), for which, using geometric quantification (logarithmic quantification of φ), the role of central frequency will be played in this connection by the frequency $\varphi = 2.0$ Hz. This interval is quite credibly relevant. Larger intervals were believed to be less appropriate, due to data processing problems.

Returning now to the problem of a possible change of the parameter b, it is clear that a possible change will lead to a change of the estimated intensity values. It is assumed that a possible change of b will be undertaken under the condition that a certain, reference, intensity will be kept unchanged. Two logarithm bases, b' and b'', and two corresponding free terms, I_{X0}' and

I_{X0}'' respectively, are considered for relation (1.a). Their use would lead to different estimated intensities, I_X' and I_X'' respectively, excepted a certain „control" intensity I_X' $= = I_X'' = I_{Xc}$. In case one wants the two estimates to coincide for the reference intensity $I_X = I_{Xc}$, the conditions.

$$I_{Xc} = \log_{b'} Q_{Xc} + I_{X0}' = I_{XQ}' + I_{X0}' = \log_{b''} Q_{Xc} + I_{X0}'' =$$
$$= I_{XQ}'' + I_{X0}'' \qquad (13)$$

are to be fulfilled. This leads to the result (for the quantification of the new intensity I_{X0}'')

$$I_{X0}'' = I_{Xc} - (I_{Xc} - I_{X0}') \times (\lg b' / \lg b'') \qquad (14)$$

(lg: decimal logarithm).

Homologous relations should be used for i_x too. An additional problem to be considered is that of estimating *EPAS* and *EPVS* on the basis of using as input data the intensity I_S, (1.a), (3), and the velocity / acceleration corner frequency φ_c (4). This leads to the expressions

$$EPAS\,(m/s^2) = [b\uparrow (I_S - I_{S0}) \times (2\,\pi\,\varphi_c)]^{1/2} \qquad (15.a)$$

$$EPVS\,(m/s) = [b\uparrow (I_S - I_{S0}) / (2\,\pi\,\varphi_c)]^{1/2} \qquad (15.b)$$

Previous developments make it possible to build an expression of a design spectrum (in case design intensity and corner frequency are specified), at least in the neighborhood of the velocity / acceleration corner frequency φ_c.

$$s_a^*(\varphi)\,(m/s^2) = 2.5 \times [(2\,\pi\,\varphi_c) \times b\uparrow (I_S - I_{S0})]^{1/2}$$
$$(\varphi \geq \varphi_c) \qquad (16.a)$$

$$s_a^*(\varphi)\,(m/s^2) = 2.5 \times [(2\,\pi\,\varphi_c) \times b\uparrow (I_S - I_{S0})]^{1/2} \times (\varphi_c / \varphi)$$
$$(\varphi < \varphi_c) \qquad (16.b)$$

b) Statistical Analysis and Parameter Calibration

The strong earthquakes of Romania of 1977, 1986 and 1990 provided a quite rich database of accelerograms, and this was used in order to investigate r.m.s. deviations and correlations between the various intensities: global intensities I_{XQ} and averaged intensities $i_{xq}^{\sim}(\varphi', \varphi'')$ introduced in equations (1). Subsequent calibration of parameters I_{X0} and i_{x0} was conducted on this basis [Sandi & Floricel, 1998].

The *primary* processing concerned:

- the global quantities Q_S, Q_A (note relation (8) too);
- the frequency dependent quantities $q_s(\varphi)$, $q_d(\varphi)$, $q_f(\varphi)$ determined for 121 φ values each (the values φ represented practically a geometric progression in the frequency interval (0.25 Hz, 16.0 Hz);
- the averaged values $q_s^{\sim}(\varphi', \varphi'')$, $q_d^{\sim}(\varphi', \varphi'')$, $q_f^{\sim}(\varphi', \varphi'')$, determined alternatively for the following frequency intervals (φ', φ''): (0.25, 16.), (0.5, 8.), (1, 4.), (0.25, 0.5), (0.5, 1.0), (1.0, 2.0), (2.0, 4.0), (4.0, 8.0), (8.0, 16.0), where the numerical values are expressed in Hz.

The quantities I_{XQ}, $i_{xq}(\varphi)$ and $i_{xq}^{\sim}(\varphi', \varphi'')$ were determined thereafter. They served as a basis for graphic representations as well as for correlation and regression analysis.

The *secondary processing* was related to correlation and regression analysis. Following combinations were considered:

(a) $I_S \leftrightarrow I_A$, $I_S \leftrightarrow i_s^{\sim}(\varphi', \varphi'')$, $I_S \leftrightarrow i_d^{\sim}(\varphi', \varphi'')$, $I_S \leftrightarrow i_f^{\sim}(\varphi', \varphi'')$, where (φ', φ'') was (0.25 Hz, 16. Hz);

(b) $I_A \leftrightarrow i_s^{\sim}(\varphi', \varphi'')$, $I_A \leftrightarrow i_d^{\sim}(\varphi', \varphi'')$, $I_A \leftrightarrow i_f^{\sim}(\varphi', \varphi'')$, where (φ', φ'') was the same;

(c) $i_s^{\sim}(\varphi', \varphi'') \leftrightarrow i_d^{\sim}(\varphi', \varphi'')$, $i_s^{\sim}(\varphi', \varphi'') \leftrightarrow i_f^{\sim}(\varphi', \varphi'')$, $i_d^{\sim}(\varphi', \varphi'') \leftrightarrow i_f^{\sim}(\varphi', \varphi'')$, where (φ', φ'') was the same.

(d) the same as a), where (φ', φ'') was alternatively: (0.5 Hz, 8. Hz), (1. Hz, 4. Hz), (0.25 Hz, 0.5 Hz), (0.5 Hz, 1. Hz), (1. Hz, 2. Hz), (2. Hz, 4. Hz), (4. Hz, 8. Hz), (8. Hz, 16. Hz).

The variants (a), (b), (c) were intended to explore the quantities considered for a global characterization of ground motion, while the variant (d) was intended to go into details for relatively narrow (one – octave) frequency intervals.

The best correlation appeared for the control combination $I_A \leftrightarrow i_d^{\sim}$ (0.25 Hz, 16.0 Hz), for which the

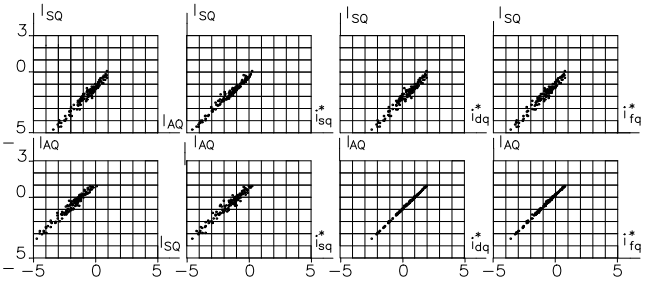


Figure 1 : Correlation of I_{SQ} and I_{AQ} between themselves and with frequency dependent parameters, averaged for the interval (0.25 Hz, 16.0 Hz)

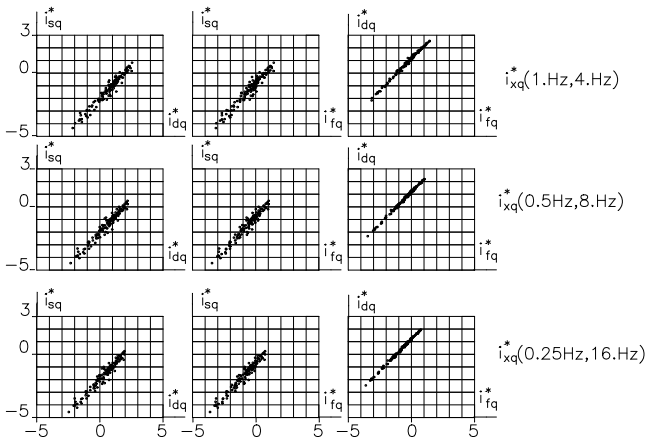


Figure 2 : Correlation between $i\tilde{}_{sq}(\varphi', \varphi'')$, $i\tilde{}_{dq}(\varphi', \varphi'')$ and $i\tilde{}_{fq}(\varphi', \varphi'')$ for various intervals (φ', φ'')

correlation coefficient was 1.00 and the r.m.s. deviation was 0.02…0.03. The weakest correlation appeared for the combination $i_s\tilde{}$ (0.25 Hz, 16.0 Hz) ↔ $i_f\tilde{}$ (0.25 Hz, 16.0 Hz), for which the correlation coefficient was 0.92 … 0.97 and the r.m.s. deviation was 0.16…0.23 (see Fig. 1, 2).

The analysis of correlation of various averaged intensities $i_x\tilde{}$ (φ', φ'') upon successive 6 dB intervals led to the results of Table 2. It showed that the best correlation exists for the frequency interval (0.25 Hz, 0.5 Hz) and this tends to decrease monotonically for intervals of increasing frequencies, up to the interval (8.0 Hz. 16.0 Hz), where it is lowest. The margins were from 0.96 …0.98 to 0.84 … 0.95 for the combination $i_{sq}\tilde{}$ ↔ $i_{dq}\tilde{}$ (strongest), from 0.92 … 0.95 to 0.52 … 0.78 for the combination $i_{sq}\tilde{}$ ↔ $i_{fq}\tilde{}$ (weakest) and from 0.98 … 1.00 to 0.78… 0.88 for the combination $i_{dq}\tilde{}$ ↔ $i_{fq}\tilde{}$.

Looking at the results of statistical analysis as a whole, it may be stated that the alternative measures of intensity introduced are quite well correlated, and this may be accepted as a strong argument in their favor.

In order to calibrate the free terms I_{X0} and i_{x0} of equations (1), it was decided to postulate one of them and then to calibrate the others in a way to lead to a best correlation

Table 2 : Correlation Coefficients for Various Frequency Intervals

(φ', φ''), Hz	$i_{sq}^* \leftrightarrow i_{dq}^*$	$i_{sq}^* \leftrightarrow i_{fq}^*$	$I_{dq}^* \leftrightarrow i_{fq}^*$
(0.25, 0.5)	0.96…0.98	0.95…098	0.98…1.00
(0.5, 1.0)	0.96…0.98	0.94…0.99	0.99…1.00
(1.0, 2.0)	0.94…0.98	0.92…0.98	0.99…1.00
(2.0, 4.0)	0.92…0.98	0.86…0.96	0.98…0.99
(4:0, 8.0)	0.91…0.96	0.82…0.86	0.95…0.97
(8.0, 16.0)	0.84…0.95	0.52…0.78	0.78…0.88

for the intensities I_X and $i_x\tilde{}$ (0.25 Hz, 16.0 Hz). The value postulated was I_S = 8.0 for the record of Bucharest – INCERC of 1977.03.04. The system of free terms (rounded up to a multiple of 0.05) is that of Table 3.

Table 3 : Calibrations Adopted for Free Terms I_{x0} and i_{x0}

Parameter	I_{S0}	I_{A0}	i_{s0}	i_{d0}	i_{f0}
Calibration	8.0	6.75	7.70	5.75	6.95

V. Some Illustrative Results

A first attempt to look at the global intensities I_S assessed for some relevant, strong, ground motions, was provided by the data of [Sandi, 1986]. Intensities I_S, determined on the basis of response spectra, were presented there for several cases of strong ground motion of Mexico, Romania, USA and former Yugoslavia. It may be stated that the agreement between I_S and macroseismic intensity estimates was at least fair. Given the strong correlation between the alternative measures I_X and $i_x\tilde{}$ (φ', φ''), the favorable conclusions on the compatibility of macroseismic estimates with the global measure I_S, this compatibility should extend to the other measures introduced.

A few illustrative results will help to better understanding of the proposals developed.

A first presentation is concerned with two, by now classical, quite frequently referred to, strong motion records: the El Centro record obtained during the Imperial Valley earthquake of 1940.05.18 and the SCT (Segretería de Comunicaciones y Transportes, Mexico City) record obtained during the Guerrero-Michoacán (Mexico) earthquake of 1985.09.19 [Borcia et al., 2012]. Both records concern high severity motions, but there exists an important difference between them, due especially to the strongly different spectral contents of ground motion. While the El Centro record is characterized by rather high dominant frequencies (as usual), the SCT record is characterized by unusually low dominant frequencies. More cases are presented in this view in [Sandi et al., 2010a] and [Sandi & Borcia, 2011]. The outcome of processing of the averaged intensity spectra $i_s\tilde{}$ (φ', φ'') and $i_d\tilde{}$ (φ', φ'') shows that the differences are minor, generally not exceeding a quarter of an intensity degree.

The shapes of response spectra for absolute acceleration, relative velocity and relative displacement can be compared directly with the averaged intensity spectra $i_s\tilde{}$ (φ', φ'') and $i_d\tilde{}$ (φ', φ''), determined for various 6 dB frequency intervals (φ', φ''). A look at the El Centro results of Fig. 3 shows that intensities were highest for oscillation periods less than 1 s, i.e. the ground motion should have affected most severely relatively rigid buildings, like those with steel frame structures with less than 10 stories, or bearing wall buildings having less than 20 stories. A similar look at the SCT results of Fig.

4 reveals a strongly different picture, since the most severe spectral zone is now in the range of periods exceeding 1 s and, especially, of periods exceeding 2 s. As it is well known, the heaviest toll of that earthquake was related to the collapse of numerous taller buildings. The intensities are about the same along the two horizontal directions for the El Centro case, but there are differences exceeding half intensity degree between the two horizontal directions in the SCT case, and this means in the latter case a quite relevant ground motion directionality. The various ground motion characteristics referred to, due to the records, are presented in Figures 3 and 4 according to the scheme of Table 4.

Table 4 : Scheme of Pictures Concerning the Illustrative Processing for the Reference Records Used

Accelerogram along the longitudinal direction	Accelerogram along the transversal direction
Response spectra for absolute accelerations for horizontal directions. Abscissa: period, natural scale.	Response spectra for absolute accelerations for horizontal directions. Abscissa: period, logarithmic scale.
Response spectra for relative velocities for horizontal directions. Abscissa: period, natural scale.	Response spectra for relative displacements for horizontal directions. Abscissa: period, natural scale.
Averaged intensity spectra (6 dB intervals): $i_s^{\sim}(\varphi', \varphi'')$ (red) and $i_d^{\sim}(\varphi', \varphi'')$ (blue) for horizontal directions. Abscissa: period, logarithmic scale.	Averaged intensity spectra (6 dB intervals): $i_s^{\sim}(\varphi', \varphi'')$ (red) and $i_d^{\sim}(\varphi', \varphi'')$ (blue) for horizontal plane. Abscissa: period, logarithmic scale.

It may be stated that the outcome of processing, represented by the averaged intensity spectra, is in fair agreement with the effects observed during post-earthquake surveys. This is obvious especially for the effects of the 1985.09.19 earthquake in the central zone of Mexico City, for which the shape of intensity spectra in the range of periods T exceeding 1 s, is in agreement with the large number of taller buildings that collapsed

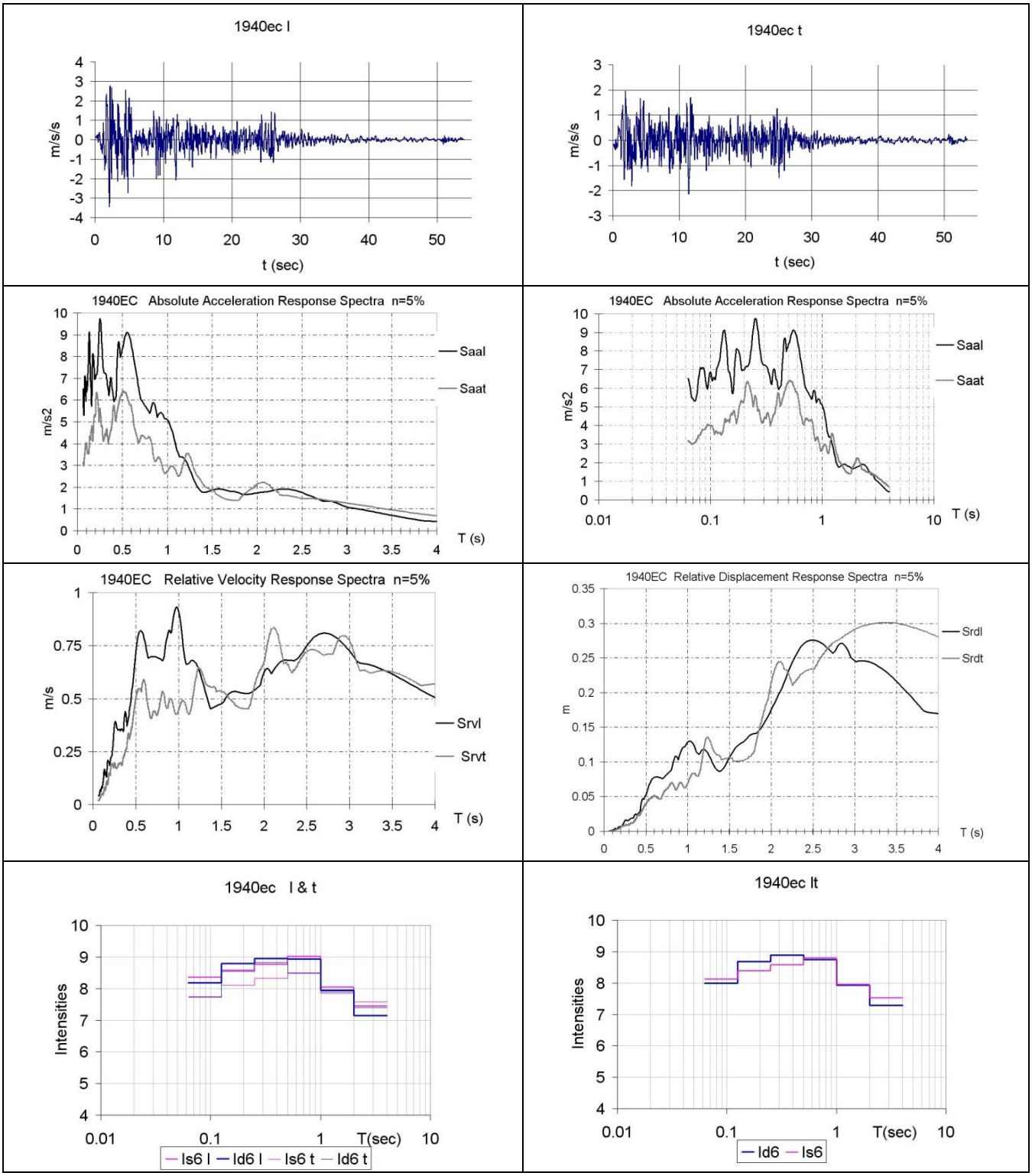


Figure 3 : Results of processing for the El Centro record of 1940.05.18

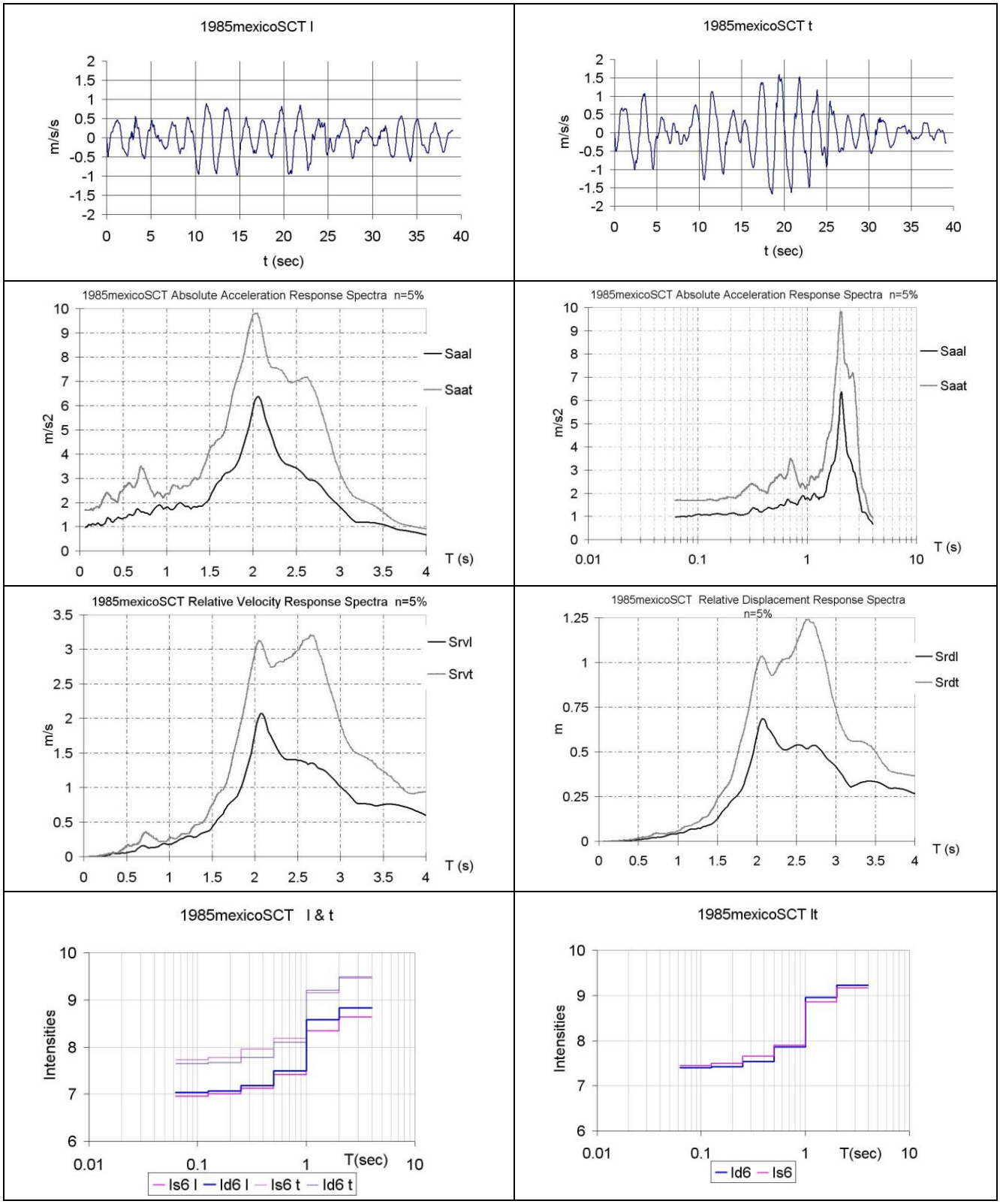


Figure 4 : Results of processing for the SCT, Mexico City, record of 1985.09.19

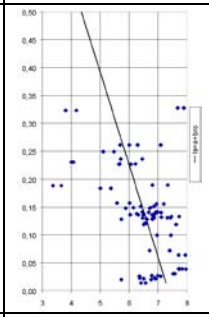
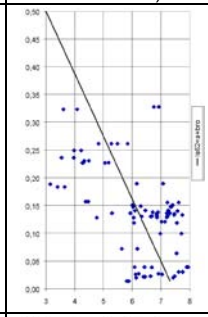
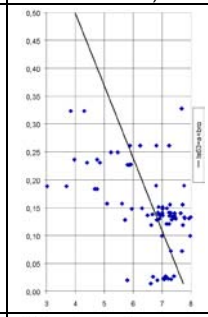
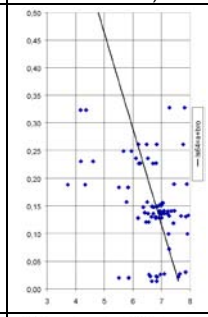
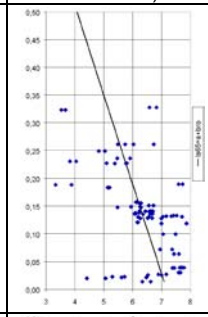
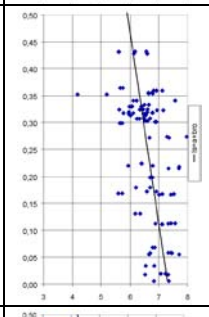
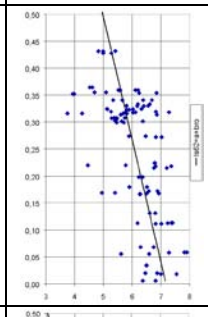
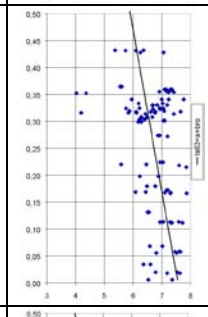
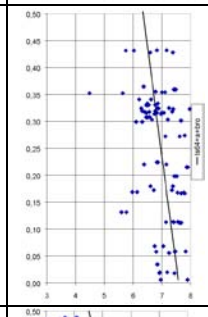
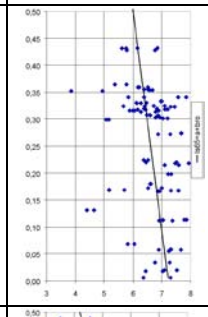
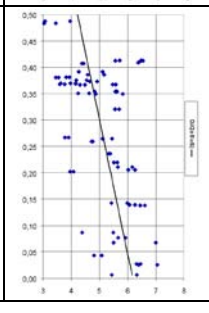
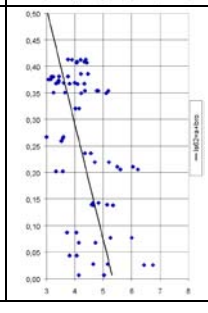
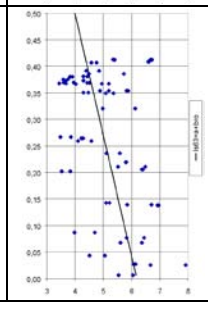
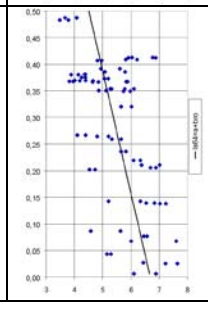
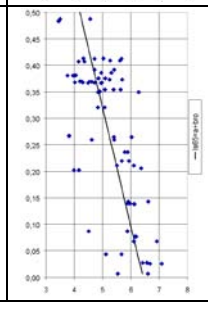
Events	I_S	$i_s^{\sim}{}_{62}$ (0.5 Hz, 1.0 Hz)	$i_s^{\sim}{}_{63}$ (1.0 Hz, 2.0 Hz)	$i_s^{\sim}{}_{64}$ (2.0 Hz, 4.0 Hz)	$i_s^{\sim}{}_{65}$ (4.0 Hz, 8.0 Hz)
1986.08.30, M_{GR} = 7.0					
1990.05.30, M_{GR} = 6.7					
1990.05.31, M_{GR} = 6.1					

Figure 5 : Regression lines for global intensities and for intensities averaged upon various frequency bands for various events and frequency bands

The cases of the El Centro and SCT records, dealt with previously, pertain to a more comprehensive analysis, which was concerned with 54 records of North America, Romania and Republic Moldova. It may be mentioned that the outcome of that investigation made it possible to compare five categories of results, concerning the macroseismic intensity and the values I_S, I_A, $i_s^{\sim}$ (0.25 Hz, 16.0 Hz) and $i_d^{\sim}$ (0.25 Hz, 16.0 Hz). It turned out that I_A and $i_d^{\sim}$ (0.25 Hz, 16.0 Hz) are in general better correlated between themselves and also with macroseismic intensity, than the homologous couple I_S and $i_s^{\sim}$ (0.25 Hz, 16.0 Hz). This confers them, of course, increased credibility.

On the other hand, it turned out that the deviations between instrumental and macroseismic intensity estimates exceeded half degree of intensity in 9% of cases only.

A second presentation concerns the analysis of the phenomenon of radiation / attenuation, expressed in terms of various intensities, I_S and (φ', φ''), for the strong Vrancea, Romania, earthquakes of 1986.06.30 (M_{GR} =7.0, M_w = 7.3), 1990.05.30 (M_{GR} = 6.7, M_w = 7.0) and 1990.05.31 (M_{GR} = 6.1, M_w = 6.4). A first approach, presented in Fig. 5, is related to the analysis of this phenomenon irrespective of azimuthal direction. The successive columns concern the global intensity I_S and the intensities $i_s^{\sim}$ (φ', φ''), averaged for motion in the horizontal plane, for the successive 6 dB intervals (φ', φ'') ranging from (0.5 Hz, 1.0 Hz) to (4.0 Hz, 8.0 Hz). The regression lines are plotted against the clouds of local intensities estimated for the various recording stations. A second approach, presented in Fig. 6, is related to the analysis of the phenomenon paying attention also to the azimuthal direction of investigation. A Fourier analysis with respect to the azimuthal direction, performed in statistical terms, made it possible to determine the distances up to which the intensities of 5.0, 6.0 and 7.0 respectively, are likely to have occurred. The global intensities I_S, and the intensities $i_s^{\sim}$ (φ', φ''), averaged for the successive 6 dB intervals (φ', φ'') ranging from (0.5 Hz, 1.0 Hz) to (4.0 Hz, 8.0 Hz), were used for plotting. One of the most interesting results is the fact that, while the dominant radiations direction were rather similar for

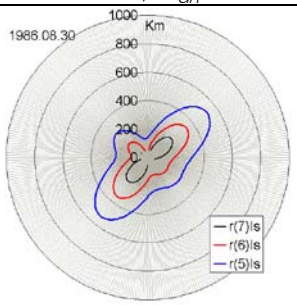
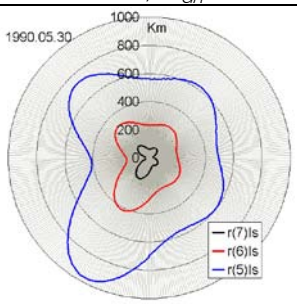
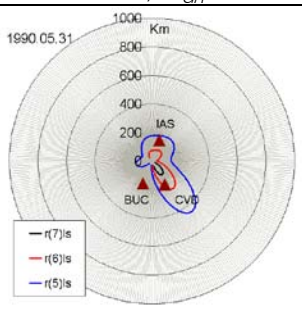
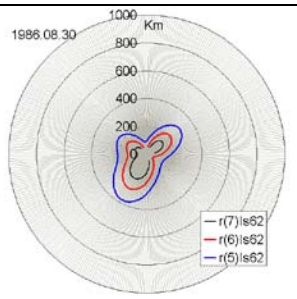
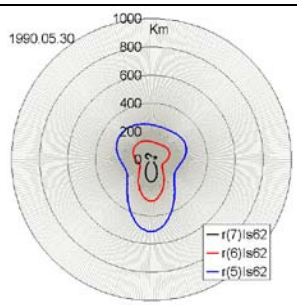
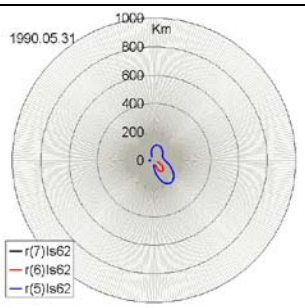
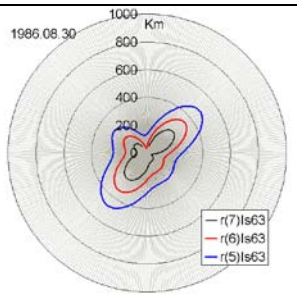
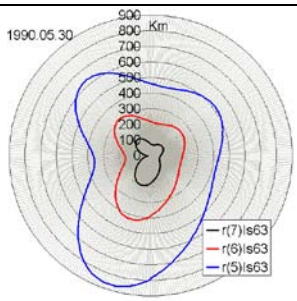
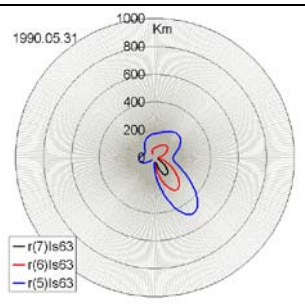
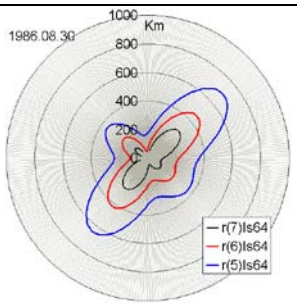
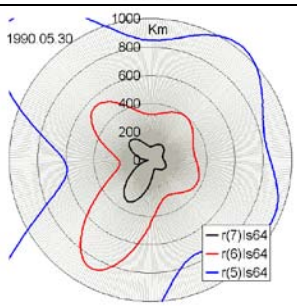
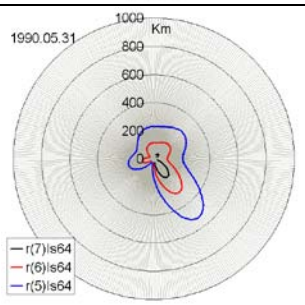
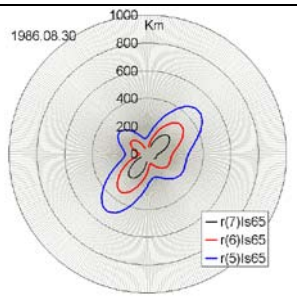
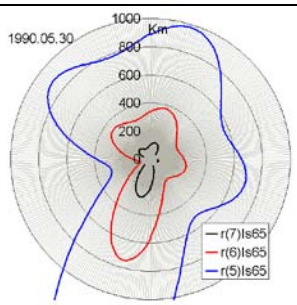
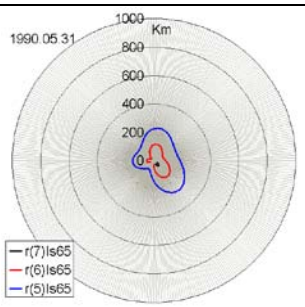
Intensities	1986.08.30, M_{GR} = 7.0	1990.05.30, M_{GR} = 6.7	1990.05.31, M_{GR} = 6.1
I_S			
$i_s^{\sim}{}_{62}$ (0.5 Hz, 1.0 Hz)			
$i_s^{\sim}{}_{63}$ (1.0 Hz, 2.0 Hz)			
$i_s^{\sim}{}_{64}$ (2.0 Hz, 4.0 Hz)			
$i_s^{\sim}{}_{65}$ (4.0 Hz, 8.0 Hz)			

Figure 6 : Directionality of radiation / attenuation, for various events and frequency bands (common scale, up to epicentral distance of 1000 km)

the first two events (as usual for strong Vrancea events), they were strongly different for the third one.

On the other hand, one may remark that the dominant radiation directions may be nevertheless different for different spectral bands (see event of 1990.05.30).

VI. Final Considerations

The experience gathered from the use of concepts developed and of the intensity measures proposed makes it possible to derive some conclusions and recommendations.

The system proposed appears to be flexible, in the sense that the user can adopt solutions providing more or less information, according to user needs.

While traditional intensity degrees are discrete and offer no information on spectral contents or on directionality of motion, the system proposed makes it possible to obtain, and subsequently to use, much more information, depending on needs.

The system proposed appears to be compatible with the consideration of macroseismic information. In case of discrepancies, one should rather look for possible distortions due to macroseismic surveys, as illustrated by the experience of Romania, referred to in Section 2.

A first recommendation derived for conducting post-earthquake field surveys is concerned with the need of consideration of the implications of the spectral content of ground motion. The main requirement in this view is to identify the spectral domain for which the earthquake effects observed are relevant. Since, in the range of intensities in which we are the most interested, namely that of severe ground motions producing damage to the artifacts of man (basically for a spectral band of about (0.25 Hz, 16.0 Hz)), when damage is investigated one should also examine to which more narrow spectral band the relevant dynamic characteristics of works affected pertain. In terms of measures presented previously, to identify the frequency band (φ', φ'') for which the intensity $i_x^{\sim}(\varphi', \varphi'')$, believed to have been observed, should be relevant. This requirement should be considered for completing the methodology as well as the forms to be used in post-earthquake field surveys.

The intensity measures mostly used by the author were I_S and I_A for global intensities on one hand and $i_s^{\sim}(\varphi', \varphi'')$ and $i_d^{\sim}(\varphi', \varphi'')$ for averaged intensities on the other hand. It turned out that I_S is quite easy to use: after some exercise, looking at a response spectrum makes it possible, by mental calculations, to get a quite precise idea on the corresponding intensity. This makes it most useful for a first estimate. On the other hand, the couple of measures I_A and $i_d^{\sim}(\varphi', \varphi'')$ appears to be more stable and better correlated with macroseismic estimates (besides the advantage of being appropriate for in depth directionality investigation). This appears to make that couple well suited for detailed, in depth, analyses.

The problem of the logarithm base b, to be used, was raised in Section 4. This is yet an open question. An attempt [Borcia et al., 2010] to derive conclusions in this respect, comparing the outcome of alternative use of the values $b = 4.0$ or $b = 7.5$ for a sample of 54 strong motion records of North America, Romania and Republic Moldova did not provide clear arguments in favour of the use of one or the other of the values considered. While the structure of equations (1) appeared to be satisfactory, the adoption of a most appropriate value for the base b may thus remain a task of further research.

Another question, yet open, is represented by the concern about the way of consideration of the vertical component of ground motion. This should also be dealt with in future.

The case studies presented in Section V illustrate the variety of problems that can be investigated by means of the tools developed. Of course, other categories of problems to be analyzed by means of the use of the system can be identified too.

In case the drafting of a regulatory document describing the instrumental scale proposed is initiated, the instrumental criteria developed should be postulated to be the basic ones, while macroseismic criteria (completed with specifications concerning the spectral content and calibrated to be most compatible with instrumental criteria) should become secondary ones

VII. Acknowledgements

The author would like to express his deep gratitude to his colleagues Ioan Sorin Borcia and Ion Floricel, for their dedication and valuable contribution to research in this field, especially in connection with the elaborate processing of instrumental data.

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On the Calculation of Crack Width in RC Linear Elements under Eccentric Load

By A. Pisanty & R. Farhat
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Keywords : concrete structures, structural design, crack control, crack width calculation, steel reinforcement, consti-tutive laws, serviceability limit state.

GJRE-E Classification : FOR Code: 290899p, 680302

Strictly as per the compliance and regulations of :

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A. Pisanty [α] & R. Farhat [σ]

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I. Notation

A_s - the area of reinforcement close to the tension face of the section
A_s' - the area of reinforcement close to the compres-sion face of the section
d - effective height of the section
d_s - distance from the center of the tensile rein-forcement As to the extreme fiber in tension
d_s' - distance from the center of the compression reinforcement As' to the extreme fiber in com-pression
e_d - eccentricity of the normal force relative to sec-tion center
E_{cm}- concrete modulus of elasticity
E_s - reinforcing bars modulus of elasticity
f_{ctm} - the mean tensile strength of the concrete
f_{yk} - yield strength of reinforcing bars
K_1-k_2 – coefficients for calibration of S_{rm} (bond and stress distribution) [ENV 1992-1-1:dec. 1991]
M_{dser}- service moment acting on the section result-ing from static analysis
$M_{sd,ser}$ - moment acting on the section after normal force being transferred to A_s
$N_{d,ser}$ - service normal force acting on the section resulting from static analysis
s_{rm} - average final crack spacing [ENV 1992-1-1:dec. 1991]
w_k - the design crack width
y - distance from the extreme fiber in tension to the section center
y' - distance from the extreme fiber in compression to the section center
β - coefficient relating the average crack width to the design crack width [ENV 1992-1-1:dec. 1991]
ε_{sm} - mean strain in the reinforcement at the crack allowing for tension stiffening
ϕ - bar's diameter (or the average scaled bars diameters)
ρ_r - reinforcement ratio relative to the effective concrete section in tension Ac,eff.
σ_{sr} - stress in the tensile reinforcement under the cracking moment Mcr [ENV 1992-1-1:dec. 1991]
σ_s - stress in the tensile reinforcement under the service moment including the axial force transferred to the tensile reinforcement

II. Introduction

Limiting crack width is one of the two basic condi-tions (but not only) for securing suitable perfor-mance in serviceability limit state: deformation and cracking limitation. The later is no less important since crack width requirements are more relaxed than in the past, but the need of verification is essen-tial. Some codes, like the ACI [ACI 318M-05], have given up calculating crack width, assuming that con-trol may be attained indirectly. The EN 2 in its for-mer [ENV 1992-1-1:dec. 1991] and present [BS EN 1992-1-1:2004] versions, has pursued in providing procedures for calculating the crack width, however, focusing on pure bending mainly. Considering ec-centrically loaded sections is important in both RC and PC elements. A simple procedure is offered here that allows a straightforward crack width calculation in linear concrete elements, eccentrically loaded. The results are compared with the limitations im-posed by EN2 [1992, 2004] and with the stress state of sections eccentrically loaded obtained by nonli-near material analysis [Farhat, R., 1995] and found to be in very good agreement.

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III. Calculation of Crack Width According to En 2 [ENV 1992]

EN 2 [ENV 1992] offered the following procedure for calculating crack width:

$$w_k = \beta\, s_{rm}\, \varepsilon_{sm} \tag{1}$$

The average final crack spacing defined as:

$$s_{rm} = 50 + 0.25\, k_1\, k_2\, \phi/\rho_r \tag{2}$$

Note: k_2 = 0.5 for bending and 1.0 for pure tension with possible interpolation for intermediate cases according to:

$k_2 = (\varepsilon 1 + \varepsilon 2)/2\ \varepsilon 1$ with $\varepsilon 1$ & $\varepsilon 2$ being the greater and the lesser tensile strains at the boundaries of the sec-tion considered (quote).

Though this definition leaves the impression that ec-centric load is dealt with, it appears not to be the case, as eccentric compression is not included in this consideration and in a cracked section under eccen-tric tension there hardly is any possibility to calcu-late ε_1, while undoubtedly ε_2 will be in compression.

The mean strain in the reinforcement defined as:

$$\varepsilon_{sm} = \frac{\sigma_s}{E_s}\left[1-\beta_1\beta_2\left(\frac{\sigma_{sr}}{\sigma_s}\right)^2\right] \tag{3}$$

This procedure EN 2 [ENV 1992] was modified in EN 2 [BS EN 1992-1-1:2004] to:

$$w_k = s_{r,max}\,(\varepsilon_{sm} - \varepsilon_{cm}) \tag{4}$$

Essentially there is difference in the cracks spacing and the strains, however the final calculation results according both renders almost identical results.

IV. Proposed Method for Calculation of Crack Width Under Eccentric Load

The proposed herein method, follows the procedures as given in [ENV 1992-1-1:dec. 1991] (detailed above) or [BS EN 1992-1-1:2004], except for a transformation suggested that allows for easy and simple consideration of the eccentricity in loading. Only the procedure given in EN2 [ENV 1992-1- 1:dec. 1991] is discussed in the following, however in the numerical examples that follow crack width is calculated according both EN2 versions.

A symmetrical with reference to vertical axis section is given in Figures 1a&2a (see notation). On the section acts a normal force in service $N_{d,ser}$ at eccen-tricity e_d vs. the section center, as obtained from elastic static analysis. The case of $N_{d,ser}$ in compres-sion with e_d is given at Figure 2a and $N_{d,ser}$ in tension with e_d is given in Figure 1a.

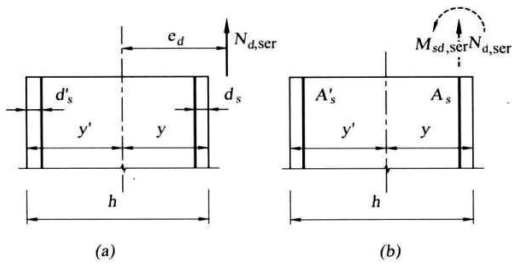

Figure 1 : Eccentric normal force in tension acting on a section

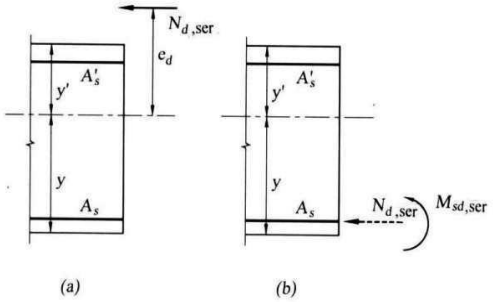

Figure 2 : Eccentric normal force in compression acting on a section

1. It is proposed to transfer the load to the center of the tensile (or the less compressed) reinforcement in the section - As. In order to maintain equili-brium, after transfer, the moment will be:

$$M_{sd\,ser} = N_{d\,ser}\,[e_d + (y - d_s)] \tag{5}$$

for eccentric compression - see Fig. 2b

$$M_{sd,ser} = N_{d,ser}\,[e_d - (y - d_s)] \tag{6}$$

for eccentric tension - see Fig. 1b

From here on the section analysis for cracking will be conducted under the action of $M_{sd,ser}$ and $N_{d,ser}$.

2. The stress in the tensile face is to be checked as-suming uncracked section. If it exceeds f_{ctm} (the mean tensile strength of the concrete) the section is cracked.
3. The stress in the tensile reinforcement will be:

for eccentric tension

$$\sigma_s = \frac{M_{sd,ser}}{0.87\,d\,A_s} + \frac{N_{d,ser}}{A_s} \tag{7}$$

$$\sigma_s = \frac{M_{sd,ser}}{0.87\,d\,A_s} - \frac{N_{d,ser}}{A_s} \tag{8}$$

for eccentric compression

4. The stress σ_{sr} - in the tensile reinforcement under the cracking moment M_{cr} is (ignoring the normal force):

$$\sigma_{sr} = \frac{M_{cr}}{0.87\, d\, A_s} \tag{9}$$

5. The average strain in the tensile reinforcement is calculated as given in (3) above. β_1 and β_2 remain as recommended there.
6. The average distance between cracks srm is calculated according to (2) above, with k1 = 0.8 for high bond bars and k_2 = 0.5 for pure bending. ϕ and ρr as defined in EN2 [ENV 1992].
7. Finally the maximum crack width, according to EN2 [ENV 1992] is:

$$w_{max} = 1.7\, s_{rm}\, \varepsilon_{sm} \tag{10}$$

V. Numerical Examples

The examples given in the following aim to cover a variety of problems that may rise applying the of-fered procedure. In all examples the concrete type is f_{ckcyl}=25Mpa with mean concrete tensile strength - f_{ctm} = 2.6 MPa and E_{cm} = 31000 MPa. The reinforcement consists of ribbed single bars (ϕ) with f_{yk} = 400 MPa and/or welded mats of high strength welded bars (ψ) with f_{yk} = 500 MPa.

Example 1

A section of a wall, 300 mm thick, contains 2000 mm²/m tensile reinforcement in the form of Φ16@100mm at a distance d_s= 50 mm from the in-ner face of the wall, (See Figure 3).

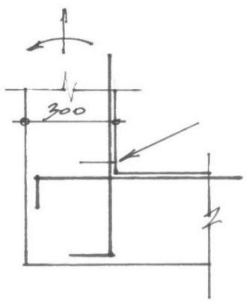

Figure 3 : Wall section, 300 mm thick, eccentrically loaded

The calculated maximum loading on the wall at this section produces:

$$M_{d,ser} = 75.3\text{kNm/m} \quad N_{d,ser} = 115.9\text{kN/m}$$

The section effective depth d is 250mm.

Solution:

The section is under tensile load with eccentricity

e_d = 75.3/115.9 = 0.65m

Transferring the load to the center of the tensile rein-forcement results in a moment:

$$M_{sd,ser} = 115.9[0.65 - (0.15 - 0.05)] = 63.75\text{kNm/m}$$

The stress at service in the tensile reinforcement will be:

$$\sigma_s = \frac{63.75\,10^6}{0.87\ 250\ 2000} + \frac{115900}{2000} = 204.5\,\text{MPa}$$

With a cracking moment M_{cr} = 39.0 kNm/m

$$\sigma_{sr} = \frac{39.0\,10^6}{0.87\ 250\ 2000} = 89.7\,\text{MPa}$$

The mean strain in the reinforcement is:

$$\varepsilon_{sm} = \frac{204.5}{2\,10^5}[1 - 0.5(\frac{89.7}{204.5})^2] = 0.924\,10^{-3}$$

An estimate of $A_{c,eff}$ gives 80000 mm², therefore

$$\rho_r = 2000/80000 = 0.025$$

The average distance between cracks will be:

$$s_{rm} = 50 + 0.25\ 0.8\ 0.5\ 16/0.025 = 114\,\text{mm}$$

Therefore the calculated maximum crack width is:

$$w_{max} = 1.7\ 114\ 0.924\ 10^{-3} = 0.179\ \text{mm}$$

The maximum crack width assessed according to EN2 [BS EN 1992-1-1:2004] is 0.196 mm.

The result was reviewed with the aid of:

a. Nonlinear section analysis developed by Farhat [Farhat, 1995] wherefrom the stresses and strains in the cracked section are as follows:

$$\varepsilon_c = -0.332\,10^{-3} \qquad \varepsilon_s = 0.989\,10^{-3}$$
$$\sigma_c = -7.6\text{MPa} \qquad \sigma_s = 197.8\,\text{MPa}$$

The difference between the suggested here analysis and the nonlinear analysis for σ_s is 3.3% - within very reasonable level of accuracy.

b. According to Table 7.2N [BS EN 1992-1-1:2004] for a maximum bar diameter of 16 mm and at a stress level of 200 MPa the crack width to be ex-pected will be approximately 0.2 mm. Also, accord-ing to Table 7.3N [BS EN 1992-1-1:2004] when the maximum bars spacing does not exceed 150 mm and the stress level is about 200 MPa the maximum ex-pected crack width is 0.2 mm. Here the distance be-tween the bars 100 mm therefore a crack width of less than 0.2 mm should be expected.

Example 2

A 400 mm thick section of a floor is given (Figure 4) where the tensile reinforcement at a distance 50 mm from the upper

$$\Phi 14@125mm + \#\psi 12@125mm$$

Any bottom floor reinforcement is ignored for the purpose of this analysis.

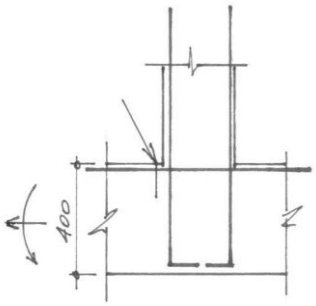

Figure 4 : Floor section, 400 mm thick, eccentrically loaded

The reinforcement placed in the form of a matt is not fully embedded in the support therefore one half of the amount is considered active, however scaling the amount in terms of strength to an equivalent of ribbed bars the total amount of reinforcement is 1760 mm^2 (1200+560).

Due to the most extreme load combination the fol-lowing was obtained:

$$M_{d,ser} = 75.9kNm/m \qquad N_{d,ser} \quad 150.9kN/m$$

Solution:

The section is under tensile load with eccentricity

e_d = 75.9/150.9 = 0.50m.

Transferring the load to the center of the tensile rein-forcement results in a moment:

$$M_{sd,ser} = 150.9[0.50 - (0.20 - 0.05)] = 52.82kNm/m$$

The stress at service in the tensile reinforcement will be :

$$\sigma_s = \frac{52.82\,10^6}{0.87\,350\,1760} + \frac{150900}{1760} = 184.2\,MPa$$

The cracking moment is 69.33 kNm/m

$$\sigma_{sr} = \frac{69.33\,10^6}{0.87\,350\,1760} = 129.4\,MPa$$

Therefore the mean strain in the reinforcement is:

$$\varepsilon_{sm} = \frac{184.2}{2\,10^5}[1 - 0.5(\frac{129.4}{184.2})^2] = 0.694\ 10^{-3}$$

An estimate of $A_{c,eff}$ gives 113300 mm^2, therefore

$$\rho_r = 1760/1133000 = 0.0155$$

The average distance between cracks will be (with an average bars diameter – 13mm):

$$s_{rm} = 50 + 0.25\ 0.8\ 0.5\ 13/0.0155 = 133.9mm$$

Therefore the calculated maximum crack width is:

$$w_{max} = 1.7\ 133.9\ 0.694\ 10^{-3} = 0.158\ mm$$

w_{max} calculated according to EN2 [BS EN 1992-1-1:2004] is 0.160 mm.

Discussion of the results:

a. Stresses and strains resulting from nonlinear section analysis [Farhat, 1995] produce:

$$\varepsilon_c = -0.203\,10^{-3} \qquad \sigma_c = -4.81MPa$$
$$\varepsilon_s = 0.883\,10^{-3} \qquad \sigma_s = 176.6\ MPa$$

Again the difference between σs from the analysis offered and the nonlinear analysis [Farhat, 1995] is 4.1% - a very fair level of accuracy.

b. According to table 7.2N [BS EN 1992-1-1:2004] for a stress level of 180 MPa in the reinforcement a maximum bar size of over 16 mm is allowed for limiting crack width to 0.2 mm.

According to Table 7.3N [BS EN 1992-1-1:2004] for the stress level of 180 MPa a maximum bar spac-ing exceeds 150 mm, but in the current example the distance is 125 mm, therefore it may be concluded that the max. crack width is lower than 0.2 mm (here – 0.158 mm).

Example 3

A portion of the ceiling of a buried underground structure is given, 400 mm thick, having

$$\phi 14@125mm + \#\psi 12@125mm$$

2320 mm^2/m at distance 50 mm from the bottom face and 1111 mm2/m at distance 50 mm from the upper face.

The effective depth d is 350 mm. See Figure 5.

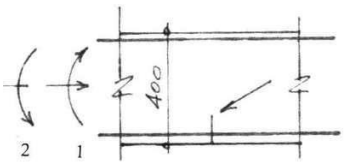

Figure 5 : The ceiling of a buried underground structure section, 400 mm thick, eccentrically loaded

Two different loading combinations are considered:

1. $M_{d,ser}$ = 120.3 kNm/m causing tension at the bottom face together with a compressive force

$$N_{d,ser} = -123.7\ kN/m$$

2. Md,ser = 30.9 kNm/m causing tension at the upper face together with a tension force

$$N_{d\ ser} = 137.7\ kN/m$$

Solution:

Addressing loading combination 1:

The section is under compressive load with eccen-tricity e_d = 120.3/123.7 = 0.973m

Transferring the load to the center of the tensile rein-forcement results in a moment:

The stress at service in the tensile reinforcement will be

$$\sigma_s = \frac{138.92\,10^6}{0.87\ 350\ 2320} - \frac{123700}{2320} = 143.3 MPa$$

The cracking moment is 69.33 kNm/m

$$\sigma_{sr} = \frac{69.33\,10^6}{0.87\ 350\ 2320} = 98.1 MPa$$

Therefore the mean strain in the reinforcement is:

$$\varepsilon_{sm} = \frac{143.3}{2\,10^5}[1 - 0.5(\frac{98.1}{143.3})^2] = 0.549\,10^{-3}$$

With:

The average distance between cracks will be:

$$A_{c,eff} = 113300\ mm^2,\ \rho_r = 2320/113300 = 0.0205$$

Therefore the calculated maximum crack width is:

$$s_{rm} = 50 + 0.25\ 0.8\ 0.5\ 13/0.0205 = 113.4 mm$$

w_{max} according to EN2 [BS EN 1992-1-1:2004] is 0.109 mm

Addressing load combination 2:

Checking stresses assuming uncracked state under eccentric tension proves that in the upper face the stress is 1.50 MPa and at the bottom face the stress is -0.82 MPa.

Discussion of the results for load combination 1:

a. The stresses and strains obtained in nonlinear analysis [Farhat, 1995] are:

$$\varepsilon_c = -0.33\,10^{-3} \qquad \sigma_c = -7.55\ MPa$$

$$\varepsilon_s = 0.69\,10^{-3} \qquad \sigma_s = 137.4\ MPa$$

The stress in the reinforcement for the proposed analysis is 143.3 MPa and the difference is again only 4.1%.

b. According to Table 7.2N [BS EN 1992-1-1:2004] at stress level 140 MPa the crack width to be expected is way below 0.2 mm. According to Table 7.3N [BS EN 1992-1-1:2004] for stress level 140 MPa the bars spacing for limiting the cracks to 0.2 mm is 200 mm but we have here only 125 mm, therefore the result obtained is acceptable.

VI. Conclusions

A simple procedure is presented; modifying slightly the proposed procedure in EN2 in it's both versions, allowing calculating directly crack width in linear concrete members, with sections under eccentric tensile or compressive normal force. Several exam-ples offered demonstrate the simplicity and practi-cality in application of the procedure. Nonlinear section analysis proves a very good cor-respondence with the results obtained with the sim-plified method offered. A good correspondence is obtained also with forecasts from EN2 [ENV 1992- 1-1:dec. 1991] and EN2 [BS EN 1992-1-1:2004] based themselves on similar calculations.

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3. Think Like Evaluators: If you are in a confusion or getting demotivated that your paper will be accepted by evaluators or not, then think and try to evaluate your paper like an Evaluator. Try to understand that what an evaluator wants in your research paper and automatically you will have your answer.

4. Make blueprints of paper: The outline is the plan or framework that will help you to arrange your thoughts. It will make your paper logical. But remember that all points of your outline must be related to the topic you have chosen.

5. Ask your Guides: If you are having any difficulty in your research, then do not hesitate to share your difficulty to your guide (if you have any). They will surely help you out and resolve your doubts. If you can't clarify what exactly you require for your work then ask the supervisor to help you with the alternative. He might also provide you the list of essential readings.

6. Use of computer is recommended: As you are doing research in the field of Computer Science, then this point is quite obvious.

7. Use right software: Always use good quality software packages. If you are not capable to judge good software then you can lose quality of your paper unknowingly. There are various software programs available to help you, which you can get through Internet.

8. Use the Internet for help: An excellent start for your paper can be by using the Google. It is an excellent search engine, where you can have your doubts resolved. You may also read some answers for the frequent question how to write my research paper or find model research paper. From the internet library you can download books. If you have all required books make important reading selecting and analyzing the specified information. Then put together research paper sketch out.

9. Use and get big pictures: Always use encyclopedias, Wikipedia to get pictures so that you can go into the depth.

10. Bookmarks are useful: When you read any book or magazine, you generally use bookmarks, right! It is a good habit, which helps to not to lose your continuity. You should always use bookmarks while searching on Internet also, which will make your search easier.

11. Revise what you wrote: When you write anything, always read it, summarize it and then finalize it.

12. Make all efforts: Make all efforts to mention what you are going to write in your paper. That means always have a good start. Try to mention everything in introduction, that what is the need of a particular research paper. Polish your work by good skill of writing and always give an evaluator, what he wants.

13. Have backups: When you are going to do any important thing like making research paper, you should always have backup copies of it either in your computer or in paper. This will help you to not to lose any of your important.

14. Produce good diagrams of your own: Always try to include good charts or diagrams in your paper to improve quality. Using several and unnecessary diagrams will degrade the quality of your paper by creating "hotchpotch." So always, try to make and include those diagrams, which are made by your own to improve readability and understandability of your paper.

15. Use of direct quotes: When you do research relevant to literature, history or current affairs then use of quotes become essential but if study is relevant to science then use of quotes is not preferable.

16. Use proper verb tense: Use proper verb tenses in your paper. Use past tense, to present those events that happened. Use present tense to indicate events that are going on. Use future tense to indicate future happening events. Use of improper and wrong tenses will confuse the evaluator. Avoid the sentences that are incomplete.

17. Never use online paper: If you are getting any paper on Internet, then never use it as your research paper because it might be possible that evaluator has already seen it or maybe it is outdated version.

18. Pick a good study spot: To do your research studies always try to pick a spot, which is quiet. Every spot is not for studies. Spot that suits you choose it and proceed further.

19. Know what you know: Always try to know, what you know by making objectives. Else, you will be confused and cannot achieve your target.

20. Use good quality grammar: Always use a good quality grammar and use words that will throw positive impact on evaluator. Use of good quality grammar does not mean to use tough words, that for each word the evaluator has to go through dictionary. Do not start sentence with a conjunction. Do not fragment sentences. Eliminate one-word sentences. Ignore passive voice. Do not ever use a big word when a diminutive one would suffice. Verbs have to be in agreement with their subjects. Prepositions are not expressions to finish sentences with. It is incorrect to ever divide an infinitive. Avoid clichés like the disease. Also, always shun irritating alliteration. Use language that is simple and straight forward. put together a neat summary.

21. Arrangement of information: Each section of the main body should start with an opening sentence and there should be a changeover at the end of the section. Give only valid and powerful arguments to your topic. You may also maintain your arguments with records.

22. Never start in last minute: Always start at right time and give enough time to research work. Leaving everything to the last minute will degrade your paper and spoil your work.

23. Multitasking in research is not good: Doing several things at the same time proves bad habit in case of research activity. Research is an area, where everything has a particular time slot. Divide your research work in parts and do particular part in particular time slot.

24. Never copy others' work: Never copy others' work and give it your name because if evaluator has seen it anywhere you will be in trouble.

25. Take proper rest and food: No matter how many hours you spend for your research activity, if you are not taking care of your health then all your efforts will be in vain. For a quality research, study is must, and this can be done by taking proper rest and food.

26. Go for seminars: Attend seminars if the topic is relevant to your research area. Utilize all your resources.

27. Refresh your mind after intervals: Try to give rest to your mind by listening to soft music or by sleeping in intervals. This will also improve your memory.

28. Make colleagues: Always try to make colleagues. No matter how sharper or intelligent you are, if you make colleagues you can have several ideas, which will be helpful for your research.

29. Think technically: Always think technically. If anything happens, then search its reasons, its benefits, and demerits.

30. Think and then print: When you will go to print your paper, notice that tables are not be split, headings are not detached from their descriptions, and page sequence is maintained.

31. Adding unnecessary information: Do not add unnecessary information, like, I have used MS Excel to draw graph. Do not add irrelevant and inappropriate material. These all will create superfluous. Foreign terminology and phrases are not apropos. One should NEVER take a broad view. Analogy in script is like feathers on a snake. Not at all use a large word when a very small one would be sufficient. Use words properly, regardless of how others use them. Remove quotations. Puns are for kids, not grunt readers. Amplification is a billion times of inferior quality than sarcasm.

32. Never oversimplify everything: To add material in your research paper, never go for oversimplification. This will definitely irritate the evaluator. Be more or less specific. Also too, by no means, ever use rhythmic redundancies. Contractions aren't essential and shouldn't be there used. Comparisons are as terrible as clichés. Give up ampersands and abbreviations, and so on. Remove commas, that are, not necessary. Parenthetical words however should be together with this in commas. Understatement is all the time the complete best way to put onward earth-shaking thoughts. Give a detailed literary review.

33. Report concluded results: Use concluded results. From raw data, filter the results and then conclude your studies based on measurements and observations taken. Significant figures and appropriate number of decimal places should be used. Parenthetical remarks are prohibitive. Proofread carefully at final stage. In the end give outline to your arguments. Spot out perspectives of further study of this subject. Justify your conclusion by at the bottom of them with sufficient justifications and examples.

34. After conclusion: Once you have concluded your research, the next most important step is to present your findings. Presentation is extremely important as it is the definite medium though which your research is going to be in print to the rest of the crowd. Care should be taken to categorize your thoughts well and present them in a logical and neat manner. A good quality research paper format is essential because it serves to highlight your research paper and bring to light all necessary aspects in your research.

Informal Guidelines of Research Paper Writing

Key points to remember:

- Submit all work in its final form.
- Write your paper in the form, which is presented in the guidelines using the template.
- Please note the criterion for grading the final paper by peer-reviewers.

Final Points:

A purpose of organizing a research paper is to let people to interpret your effort selectively. The journal requires the following sections, submitted in the order listed, each section to start on a new page.

The introduction will be compiled from reference matter and will reflect the design processes or outline of basis that direct you to make study. As you will carry out the process of study, the method and process section will be constructed as like that. The result segment will show related statistics in nearly sequential order and will direct the reviewers next to the similar intellectual paths throughout the data that you took to carry out your study. The discussion section will provide understanding of the data and projections as to the implication of the results. The use of good quality references all through the paper will give the effort trustworthiness by representing an alertness of prior workings.

Writing a research paper is not an easy job no matter how trouble-free the actual research or concept. Practice, excellent preparation, and controlled record keeping are the only means to make straightforward the progression.

General style:

Specific editorial column necessities for compliance of a manuscript will always take over from directions in these general guidelines.

To make a paper clear

· Adhere to recommended page limits

Mistakes to evade

- Insertion a title at the foot of a page with the subsequent text on the next page
- Separating a table/chart or figure - impound each figure/table to a single page
- Submitting a manuscript with pages out of sequence

In every sections of your document

· Use standard writing style including articles ("a", "the," etc.)

· Keep on paying attention on the research topic of the paper

· Use paragraphs to split each significant point (excluding for the abstract)

· Align the primary line of each section

· Present your points in sound order

· Use present tense to report well accepted

· Use past tense to describe specific results

· Shun familiar wording, don't address the reviewer directly, and don't use slang, slang language, or superlatives

· Shun use of extra pictures - include only those figures essential to presenting results

Title Page:

Choose a revealing title. It should be short. It should not have non-standard acronyms or abbreviations. It should not exceed two printed lines. It should include the name(s) and address (es) of all authors.

Abstract:

The summary should be two hundred words or less. It should briefly and clearly explain the key findings reported in the manuscript--must have precise statistics. It should not have abnormal acronyms or abbreviations. It should be logical in itself. Shun citing references at this point.

An abstract is a brief distinct paragraph summary of finished work or work in development. In a minute or less a reviewer can be taught the foundation behind the study, common approach to the problem, relevant results, and significant conclusions or new questions.

Write your summary when your paper is completed because how can you write the summary of anything which is not yet written? Wealth of terminology is very essential in abstract. Yet, use comprehensive sentences and do not let go readability for briefness. You can maintain it succinct by phrasing sentences so that they provide more than lone rationale. The author can at this moment go straight to shortening the outcome. Sum up the study, with the subsequent elements in any summary. Try to maintain the initial two items to no more than one ruling each.

- Reason of the study - theory, overall issue, purpose
- Fundamental goal
- To the point depiction of the research
- Consequences, including definite statistics - if the consequences are quantitative in nature, account quantitative data; results of any numerical analysis should be reported
- Significant conclusions or questions that track from the research(es)

Approach:

- Single section, and succinct
- As a outline of job done, it is always written in past tense
- A conceptual should situate on its own, and not submit to any other part of the paper such as a form or table
- Center on shortening results - bound background information to a verdict or two, if completely necessary
- What you account in an conceptual must be regular with what you reported in the manuscript
- Exact spelling, clearness of sentences and phrases, and appropriate reporting of quantities (proper units, important statistics) are just as significant in an abstract as they are anywhere else

Introduction:

The **Introduction** should "introduce" the manuscript. The reviewer should be presented with sufficient background information to be capable to comprehend and calculate the purpose of your study without having to submit to other works. The basis for the study should be offered. Give most important references but shun difficult to make a comprehensive appraisal of the topic. In the introduction, describe the problem visibly. If the problem is not acknowledged in a logical, reasonable way, the reviewer will have no attention in your result. Speak in common terms about techniques used to explain the problem, if needed, but do not present any particulars about the protocols here. Following approach can create a valuable beginning:

- Explain the value (significance) of the study
- Shield the model - why did you employ this particular system or method? What is its compensation? You strength remark on its appropriateness from a abstract point of vision as well as point out sensible reasons for using it.
- Present a justification. Status your particular theory (es) or aim(s), and describe the logic that led you to choose them.
- Very for a short time explain the tentative propose and how it skilled the declared objectives.

Approach:

- Use past tense except for when referring to recognized facts. After all, the manuscript will be submitted after the entire job is done.
- Sort out your thoughts; manufacture one key point with every section. If you make the four points listed above, you will need a least of four paragraphs.

- Present surroundings information only as desirable in order hold up a situation. The reviewer does not desire to read the whole thing you know about a topic.
- Shape the theory/purpose specifically - do not take a broad view.
- As always, give awareness to spelling, simplicity and correctness of sentences and phrases.

Procedures (Methods and Materials):

This part is supposed to be the easiest to carve if you have good skills. A sound written Procedures segment allows a capable scientist to replacement your results. Present precise information about your supplies. The suppliers and clarity of reagents can be helpful bits of information. Present methods in sequential order but linked methodologies can be grouped as a segment. Be concise when relating the protocols. Attempt for the least amount of information that would permit another capable scientist to spare your outcome but be cautious that vital information is integrated. The use of subheadings is suggested and ought to be synchronized with the results section. When a technique is used that has been well described in another object, mention the specific item describing a way but draw the basic principle while stating the situation. The purpose is to text all particular resources and broad procedures, so that another person may use some or all of the methods in one more study or referee the scientific value of your work. It is not to be a step by step report of the whole thing you did, nor is a methods section a set of orders.

Materials:

- Explain materials individually only if the study is so complex that it saves liberty this way.
- Embrace particular materials, and any tools or provisions that are not frequently found in laboratories.
- Do not take in frequently found.
- If use of a definite type of tools.
- Materials may be reported in a part section or else they may be recognized along with your measures.

Methods:

- Report the method (not particulars of each process that engaged the same methodology)
- Describe the method entirely
- To be succinct, present methods under headings dedicated to specific dealings or groups of measures
- Simplify - details how procedures were completed not how they were exclusively performed on a particular day.
- If well known procedures were used, account the procedure by name, possibly with reference, and that's all.

Approach:

- It is embarrassed or not possible to use vigorous voice when documenting methods with no using first person, which would focus the reviewer's interest on the researcher rather than the job. As a result when script up the methods most authors use third person passive voice.
- Use standard style in this and in every other part of the paper - avoid familiar lists, and use full sentences.

What to keep away from

- Resources and methods are not a set of information.
- Skip all descriptive information and surroundings - save it for the argument.
- Leave out information that is immaterial to a third party.

Results:

The principle of a results segment is to present and demonstrate your conclusion. Create this part a entirely objective details of the outcome, and save all understanding for the discussion.

The page length of this segment is set by the sum and types of data to be reported. Carry on to be to the point, by means of statistics and tables, if suitable, to present consequences most efficiently.You must obviously differentiate material that would usually be incorporated in a study editorial from any unprocessed data or additional appendix matter that would not be available. In fact, such matter should not be submitted at all except requested by the instructor.

Content

- Sum up your conclusion in text and demonstrate them, if suitable, with figures and tables.
- In manuscript, explain each of your consequences, point the reader to remarks that are most appropriate.
- Present a background, such as by describing the question that was addressed by creation an exacting study.
- Explain results of control experiments and comprise remarks that are not accessible in a prescribed figure or table, if appropriate.
- Examine your data, then prepare the analyzed (transformed) data in the form of a figure (graph), table, or in manuscript form.

What to stay away from

- Do not discuss or infer your outcome, report surroundings information, or try to explain anything.
- Not at all, take in raw data or intermediate calculations in a research manuscript.
- Do not present the similar data more than once.
- Manuscript should complement any figures or tables, not duplicate the identical information.
- Never confuse figures with tables - there is a difference.

Approach

- As forever, use past tense when you submit to your results, and put the whole thing in a reasonable order.
- Put figures and tables, appropriately numbered, in order at the end of the report
- If you desire, you may place your figures and tables properly within the text of your results part.

Figures and tables

- If you put figures and tables at the end of the details, make certain that they are visibly distinguished from any attach appendix materials, such as raw facts
- Despite of position, each figure must be numbered one after the other and complete with subtitle
- In spite of position, each table must be titled, numbered one after the other and complete with heading
- All figure and table must be adequately complete that it could situate on its own, divide from text

Discussion:

The Discussion is expected the trickiest segment to write and describe. A lot of papers submitted for journal are discarded based on problems with the Discussion. There is no head of state for how long a argument should be. Position your understanding of the outcome visibly to lead the reviewer through your conclusions, and then finish the paper with a summing up of the implication of the study. The purpose here is to offer an understanding of your results and hold up for all of your conclusions, using facts from your research and generally accepted information, if suitable. The implication of result should be visibly described. Infer your data in the conversation in suitable depth. This means that when you clarify an observable fact you must explain mechanisms that may account for the observation. If your results vary from your prospect, make clear why that may have happened. If your results agree, then explain the theory that the proof supported. It is never suitable to just state that the data approved with prospect, and let it drop at that.

- Make a decision if each premise is supported, discarded, or if you cannot make a conclusion with assurance. Do not just dismiss a study or part of a study as "uncertain."
- Research papers are not acknowledged if the work is imperfect. Draw what conclusions you can based upon the results that you have, and take care of the study as a finished work
- You may propose future guidelines, such as how the experiment might be personalized to accomplish a new idea.
- Give details all of your remarks as much as possible, focus on mechanisms.
- Make a decision if the tentative design sufficiently addressed the theory, and whether or not it was correctly restricted.
- Try to present substitute explanations if sensible alternatives be present.
- One research will not counter an overall question, so maintain the large picture in mind, where do you go next? The best studies unlock new avenues of study. What questions remain?
- Recommendations for detailed papers will offer supplementary suggestions.

Approach:

- When you refer to information, differentiate data generated by your own studies from available information
- Submit to work done by specific persons (including you) in past tense.
- Submit to generally acknowledged facts and main beliefs in present tense.

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Please carefully note down following rules and regulation before submitting your Research Paper to Global Journals Inc. (US):

Segment Draft and Final Research Paper: You have to strictly follow the template of research paper. If it is not done your paper may get rejected.

- The **major constraint** is that you must independently make all content, tables, graphs, and facts that are offered in the paper. You must write each part of the paper wholly on your own. The Peer-reviewers need to identify your own perceptive of the concepts in your own terms. NEVER extract straight from any foundation, and never rephrase someone else's analysis.

- Do not give permission to anyone else to "PROOFREAD" your manuscript.

- Methods to avoid Plagiarism is applied by us on every paper, if found guilty, you will be blacklisted by all of our collaborated research groups, your institution will be informed for this and strict legal actions will be taken immediately.)
- To guard yourself and others from possible illegal use please do not permit anyone right to use to your paper and files.

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Topics	Grades		
	A-B	**C-D**	**E-F**
Abstract	Clear and concise with appropriate content, Correct format. 200 words or below	Unclear summary and no specific data, Incorrect form Above 200 words	No specific data with ambiguous information Above 250 words
Introduction	Containing all background details with clear goal and appropriate details, flow specification, no grammar and spelling mistake, well organized sentence and paragraph, reference cited	Unclear and confusing data, appropriate format, grammar and spelling errors with unorganized matter	Out of place depth and content, hazy format
Methods and Procedures	Clear and to the point with well arranged paragraph, precision and accuracy of facts and figures, well organized subheads	Difficult to comprehend with embarrassed text, too much explanation but completed	Incorrect and unorganized structure with hazy meaning
Result	Well organized, Clear and specific, Correct units with precision, correct data, well structuring of paragraph, no grammar and spelling mistake	Complete and embarrassed text, difficult to comprehend	Irregular format with wrong facts and figures
Discussion	Well organized, meaningful specification, sound conclusion, logical and concise explanation, highly structured paragraph reference cited	Wordy, unclear conclusion, spurious	Conclusion is not cited, unorganized, difficult to comprehend
References	Complete and correct format, well organized	Beside the point, Incomplete	Wrong format and structuring

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