Review of Geotechnical Provisions in Indian Seismic Code is:1893 (Part 1): 2002

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GENERAL

Recently the author attended the National Seminar on "National Building Code of India-New Direction of Revision" Jointly organized by BIS and SPA New Delhi at New Delhi. During seminar it is learnt that some of codes related to building including IS 1893 are revised and about to publish and some are in draft stage. Moreover the revised NBC incorporating all these Codes will be published latest by this year end. Actually this paper presents a discussion on the provisions related to geotechnical aspects in the Indian Seismic code, IS 1893 (Part 1): 2002 [IS1893, 2002] and proposed amendment by IIT Kanpur-GSDMA for better understanding of the subject and correct application of codal provision especially related to design of foundation of structure located in Seismic Zone III and above.

1. SOIL CLASSIFICATION

Table 1 of IS 1893 (1) presents the increase in allowable bearing pressure in soils. The type of soil mainly constituting the foundation are categorized into three types (Table 1), namely

a) Type I - Rock or Hard Soil:

Well graded gravel and sand gravel and sand gravel mixtures with or without clay binder, and clayey sands poorly graded or sand clay mixtures (GB, CW, SB, SW, and SC) having N above 30, where N is the standard penetration value.

b) Type II - Medium Soil

All soils with N between 10 and 30, and poorly graded sands or gravelly sands with little or no fines (SP) with N>15

c) Type III - Soft Soil

All soils other than SP with N<10.

The above categorization is based on IS1498-1970 [IS 1498, 1970], which employs prefixes and suffixes to classify the type and subgroup as summarized in Table 2 and Table 3. These prefixes and suffixes are used as a group symbol according to the classification of the soils.

The group symbols used in Table 1 are not consistent with the soil classification according to IS: 1498-1970. For example the incorrect group symbols used in Type I soils are GB, CW, and SB. Suffix B has not been suggested in IS: 1498-1970. Also, the group symbol CW may have meant to indicate that Type I soil can be clay (C), which is well graded (W). But gradation criteria of classification soils namely well graded (W) or poorly graded (P) is used only for coarse grained soils like Gravels (G) and Sands (S). Plasticity properties are used to subgroup fine- grained soils namely silts (M) and clays (C). Hence, classification of soil type with their appropriate group symbols needs to be rewritten.

Soil profile type can be determined by either based on **Standard Penetration Test (SPT) value** and **soil classification** using **grain size distribution data** or based on **shear wave velocity.** Standard penetration test has many limitations. Apart from the testing corrections to be applied for the N value to be used in correlating it with the soil properties and hence the soil profile type, ***one of the main limitation of use of N value is to determine an appropriate value of N for layered soil*****,** especially so for the case where there is interlaying of coarse grained soils and fine-grained soils. Also, the soil profiles can and will have large variations in the areal extent. Then it becomes extremely difficult to decide upon the N value to be used for deciding the soil profile type.

Because of the limitations of this method, **it is best to use the shear wave velocity as a supplement for the standard penetration test.** The shear wave velocity of the soil can also be used to determine the soil profile type. The shear wave velocity can be measured in-situ by using several different geophysical techniques, such as the uphole, down-hole, or cross-hole methods. Other methods that can be used to determine the in-situ shear wave velocity include the seismic cone penetrometer and suspension logger [Woods, 1994; Kramer, 2003]. Though the method is more reliable in characterizing the site, considering cost of the equipment and trained personnel required for its use, ***immediate replacement of SPT method by shear wave velocity is difficult in India.*** In the course of time the method should find place in practice in India.

Even with the limitations and all the corrections that must be applied to the measured N value, the standard penetration test is probably the most widely used field test in India and elsewhere in the world. This is because it is relatively easy to use, the test is economical compared to other types of field testing, and the SPT equipment can be quickly adapted as part of almost any type of drilling rig [Day, 2002]. In view of the above discussions, the following proposal is made by IITK-GSDMA for classification of soil profile type.

a) Type I: Rock or Hard Soils

1) Well graded gravel (GW) or well graded sand (SW) both with less than

2) Well graded Gravel- Sand mixtures with or without fines (GW-SW);

3) Poorly graded Sand (SP) or clayey sand (SC), all having N above 30;

4) Stiff to hard clays having N above 16, where N is the Standard Penetration Test value.

b) Type II - Stiff Soils

1) Poorly graded sands or Poorly graded sands with gravel (SP) with little or no fines having N between 10 and 30;

2) and stiff to medium stiff fine-grained soils, like Silts of Low compressibility (ML) or Clays of Low compressibility (CL) having N between 10 and 16.

c) Type III - Soft Soils

All soft soils other than SP with N<10. The various possible soils are

1) Silts of Intermediate compressibility (MI);

2) Silts of High compressibility (MH);

3) Clays of Intermediate compressibility (CI);

4) Clays of High compressibility (CH);

5) Silts and Clays of Intermediate to High compressibility (MI-MH or CICH);

6) Silt with Clay of Intermediate compressibility (MI-CI);

7) Silt with Clay of High compressibility (MH-CH)

2. INCREASE IN ALLOWABLE BEARING PRESSURE IN SOIL

In clause cl. 6.3.5.2 and Table 1 of Indian Seismic code [IS 1893, 2002] (Table 1), two aspects come into focus

a) The increase in allowable bearing pressure varies from 25 to 50%.

b) Even the type III soils, which are considered to be of soft fine-grained soils, have an increase in allowable bearing pressure.

The allowable bearing pressure shall be determined in accordance with IS: 6403-1981 or IS: 1888-1982 Load test (Revision 1992). ***It is a common international practice to increase the allowable bearing pressure by one-third, i.e., 33%,*** while performing seismic analysis. The rationale behind this recommendation is that the allowable bearing pressure has an ample factor of safety, and thus for seismic analysis, a lower factor of safety would be acceptable. Usually, the above recommendation is appropriate for the following materials [Day, 2002]:

1) Massive crystalline bedrock and sedimentary rock that remains intact during the earthquake.

2) Dense to very dense granular soil.

3) Heavily overconsolidated cohesive soil, such as very stiff to hard clays.

The reason being that there is no significant reduction in the ultimate strengths of these materials during seismic shaking.

This one-third increase in allowable bearing pressure is not recommended for the following materials [Day, 2002]:

1) Foliated or friable rock that fractures apart during the earthquake.

2) Loose soil subjected to liquefaction or a substantial increase in excess pore water pressure.

3) Sensitive clays that lose shear strength during the earthquake.

4) Soft clays and organic soils that is overloaded and subjected to plastic flow.

The ultimate strengths of these materials reduce by appreciable amounts during the earthquake. Since the materials are weakened by the seismic shaking, the static values of allowable bearing pressure should not be increased for the earthquake analysis. In fact, the allowable bearing pressure may actually have to be reduced to account for the weakening of the soil during strong earthquake shaking.

Considering the above, Table 4 gives the recommendations for increase in the allowable bearing pressure in soils when considering the seismic loads.

3. DETERMINING SOIL PROFILE TYPE FOR IDENTIFYING THE RESPONSE SPECTRUM

The soil profile mainly constituting the local soil below the foundation required for use of response spectra is divided into three types as given in section 1.1.It is quite natural to have variation in properties of soil, and most soil deposits have both vertical as well as lateral variation of properties depending on the geomorphic forces and source of soil formation. There may be soil layers of varying properties of the similar soil type namely coarse-grained soils (Gravels, Sands or Sandy Gravels, or Gravelly Sands); fine-grained soils (Clays or Silty Clays or Clayey Silts) or there may be interlaying of coarse grained soils and fine grained soils.

The importance of local site conditions and its role on the response of structures has been well recognized. The soil and rock at a site have specific characteristics that can significantly amplify the incoming earthquake motions traveling from the earthquake source [Lew, 2001].

IS: 1893-2002 - Part 1 has acknowledged the importance of local site effects and has defined three soil profile types, which essentially are rock or hard soils (Type I), medium soils (Type II), and soft soils (Type III) (Refer Table 1). The code has suggested a design spectrum for each of these soil profile types. ***However, the code does not explain how to decide the type of soil profile to be used to select the appropriate design acceleration spectrum, given the variation of soil profile in a particular locality***. Thus, a procedure is required to arrive at the type of soil profile.

Soil profile types are to be characterized based on the average soil properties for the upper 30 m of the soil profile. Standard penetration test is a field test conducted at regular intervals in every borehole, which has a good correlation with engineering properties of soil. N values, which are corrected for overburden and dilatancy effects, are correlated with relative density and hence the angle of internal friction for coarse-grained type of soils and the undrained shear strength of fine-grained soils [Peck et al., 1974]. Relative density reflects the state of compactness of coarse-grained soils, and the undrained strength reflects the stiffness of fine-grained soils. These, in turn, field behaviour of a profile of soil. For layered soils having varying properties over the exploration depth of 30 m, the average N values are to be obtained.

4.1 LAYERED SOIL PROFILE WITH ONLY COHESIONLESS OR COHESIVE SOIL LAYERS:

If the soil profile consists of layers of only cohesive soil, then the average standard penetration for the profile under consideration is taken as the weighted average of the standard penetration values of all the layers of soil under consideration as given by Eq. 1

$$\overline{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}}$$

where
di = thickness of layer i (in m), and
Ni = the standard penetration resistance of the ith soil layer in accordance with
approved nationally recognized standards

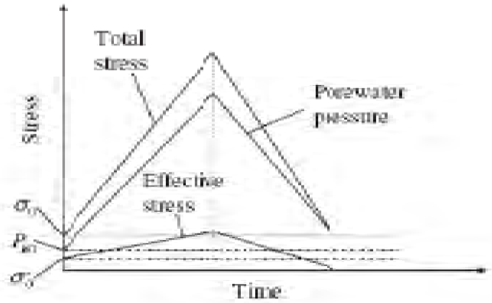

Fig. 1. Schematic of variation of total stress, porewater pressure, and effective stress during a dynamic response leading to liquefaction.

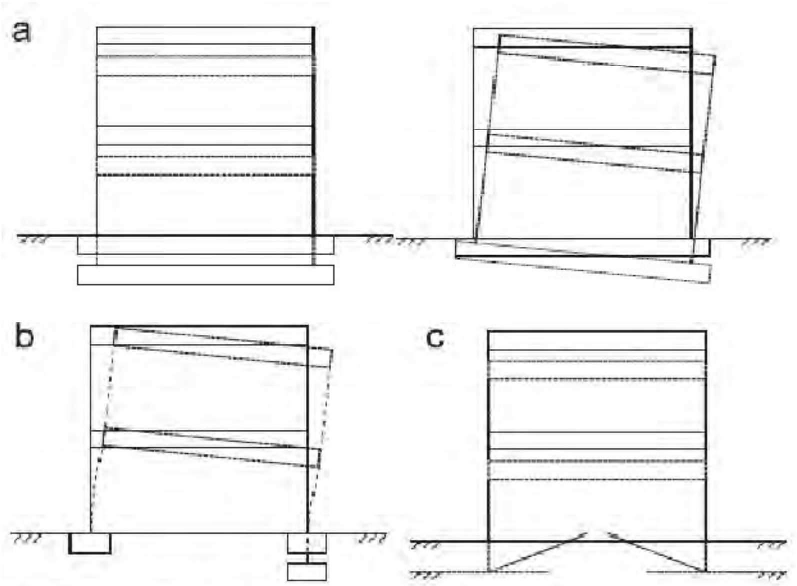

Fig. 2. Different structure settlement scenarios upon soil liquefaction (after [26]). (a) Uniform vertical settlement (left) and rigid-body rotation (right) of a structure. (b) Differential settlement. (c) Possible foundation failure.

4.2 INTER-LAYERING OF BOTH COHESION LESS AND COHESIVE SOIL LAYERS

If the soil profile consists of inter-layered layering of both cohesion less and cohesive soil layers, then the weighted average value of standard penetration for the cohesion less soil layers and cohesive soil layers is determined independently and the soil profile type (refer cl.1.1) is determined for cohesion less soil layers and cohesive soil layers separately. Then the soil profile type for the all the soil layers under consideration is decided by engineering judgment.

4. ASSESSMENT OF LIQUEFACTION POTENTIAL

As per clause 6.3.5.2 for specific type of Soil profile having N< 15 in seismic zone III, IV & V and less than 10 in seismic zone II the vibration caused by earthquake may cause liquefaction (Fig-1) or excessive total and differential settlements(Fig-2). In those areas liquefaction potential shall also be calculated along with other soil parameters. (A solved Example enclosed for ready reference).

5. CONCLUSION

Earthquakes can cause damage not only to structures on account of shaking which results from them but also due to other chain effects like liquefaction of soil .It is therefore very important to take necessary precautions in the siting, planning and design of structures so that they are safe against such secondary effects also. Although damage to buildings due to earthquake cannot be completely eliminated but the same can be minimized by taking appropriate structural measures.

Table 1: Percentage of Permissible Increase in Allowable Bearing Pressure or Resistance of Soils [IS 1893, 2002].

S. No.	Foundation	Type of soil mainly constituting the Foundation		
		Type I- Rock or Hard Soil: Well graded gravel and sand gravel and sand gravel mixtures with or without clay binder, and clayey sands poorly graded or sand clay mixtures (GB, CW, SB, SW, and SC) having N above 30, where N is the standard penetration value.	*Type II- Medium Soil:* All soils with N between 10 and 30, and poorly graded sands or gravelly sands with little or no fines (SP) with N>15	*Type III- Soft Soil:* All soils other than SP with N<10.
(1)	(2)	(3)		
i)	Piles passing through any soil but resting on soil type I	50	50	50
ii)	Piles not covered under item (i)	-	25	25
iii)	Raft foundations	50	50	50
iv)	Combined isolated RCC footing with tie beams	50	25	25
v)	Isolated RCC footing without tie beams, or unreinforced strip foundations	50	25	-
vi)	Well foundations	50	25	25

Note:

1. The allowable bearing pressure shall be determined in accordance with IS 6403 or IS 1888.
2. If any increase in bearing pressure has already been permitted for forces other than seismic forces, the total increase in allowable bearing pressure when seismic force is also included shall not exceed the limits specified above.
3. Desirable minimum field values of N – If soils of smaller N-values are met, compacting may be adopted to achieve these values or deep pile foundations going to stronger strata should be used.
4. The values of N (corrected values) are at the founding level and the allowable bearing pressure shall be determined in accordance with IS 1603 or IS 1888.

Seismic Zone level (in meters)	Depth Below Ground	N-values	Remark
III, IV and V	≤ 5	15	For values of depths between 5 m and 10 m, linear interpolation is recommended
	≥ 10	25	
II (for important structures only)	≤ 5	15	
	≥ 10	20	

5. The piles should be designed for lateral loads neglecting lateral resistance of soil layers liable to liquefy.
6. IS 1498 and IS 2131 may also be referred.
7. Isolated R.C.C footing without tie beams, or unreinforced strip foundation shall not be permitted in soft soils with N < 10.

Table 2: Prefixes of Indian Standard Soil classification System [IS 1498, 1970]

Soil Type	Prefix
Gravel	G
Sand	S
Silt	M
Clay	C
Organic	O
Peat	Pt

Table 3: Suffixes of Indian Standard Soil classification System [IS 1498, 1970]

Sub-group	Suffix
Well graded	W
Poorly graded	P
Silty	M
Clayey	C
w_L <35 %	L
35 %< w_L< 50	I
% w_L> 50 %	H

Table 4: Percentage of Permissible Increase in Allowable Bearing Pressure for Seismic Design of Foundations [IS 1893, 2002].

S.No. (1)	Foundation (2)	Type of soil mainly constituting the foundation		
		Type I	Type II	Type III
i)	Piles passing through any soil but resting on soil type I	50	50	50
ii)	Raft foundations	50	25	0
iii)	Combined isolated RCC footing with tie beams	50	25	0
iv)	Isolated RCC footing without tie beams, or unreinforced strip foundations	50	25	0
v)	Well foundations	50	25	0

Problem Statement:

The measured SPT resistance and results of sieve analysis for a site in Zone IV are indicated in Table 10.1. The water table is at 6m below ground level. Determine the extent to which liquefaction is expected for 7.5 magnitude earthquake. Estimate the liquefaction potential and resulting settlement expected at this location.

Table 10.1: Result of the Standard penetration Test and Sieve Analysis

Depth (m)	N_{60}	Soil Classification	Percentage fine
0.75	9	Poorly Graded Sand and Silty Sand (SP-SM)	11
3.75	17	Poorly Graded Sand and Silty Sand (SP-SM)	16
6.75	13	Poorly Graded Sand and Silty Sand (SP-SM)	12
9.75	18	Poorly Graded Sand and Silty Sand (SP-SM)	8
12.75	17	Poorly Graded Sand and Silty Sand (SP-SM)	8
15.75	15	Poorly Graded Sand and Silty Sand (SP-SM)	7
18.75	26	Poorly Graded Sand and Silty Sand (SP-SM)	6

Solution:

Site Characterization:

This site consists of loose to dense poorly graded sand to silty sand (SP-SM). The SPT values ranges from 9 to 26. The site is located in zone IV. The peak horizontal ground acceleration value for the site will be taken as 0.24g corresponding to zone factor Z = 0.24

Liquefaction Potential of Underlying Soil

Step by step calculation for the depth of 12.75m is given below. Detailed calculations for all the depths are given in Table 10.2. This table provides the factor of safety against liquefaction (FS_{liq}), maximum depth of liquefaction below the ground surface, and the vertical settlement of the soil due to liquefaction (Δ_v).

$$\frac{a_{max}}{g} = 0.24, M_w = 7.5,$$

$$\gamma_{sat} = 18.5\ kN/m^3,\ \gamma_w = 9.8\ kN/m^3$$

Depth of water level below G.L. = 6.00m

Depth at which liquefaction potential is to be evaluated = 12.75m

Initial stresses:

$$\sigma_v = 12.75 \times 18.5 = 235.9\ kPa$$

$$u_0 = (12.75 - 6.00) \times 9.8 = 66.2\ kPa$$

$$\sigma_v' = (\sigma_v - u_0) = 235.9 - 66.2$$
$$= 169.7\ kPa$$

Stress reduction factor:

$$r_d = 1 - 0.015z = 1 - 0.015 \times 12.75 = 0.81$$

Critical stress ratio induced by earthquake:

$$a_{max} = 0.24g,\ M_w = 7.5$$

$$CSR_{eq} = 0.65 \times (a_{max}/g) \times r_d \times (\sigma_v/\sigma_v')$$

$$CSR_{eq} = 0.65 \times (0.24) \times 0.81 \times (235.9/169.7)$$
$$= 0.18$$

Correction for SPT (N) value for overburden pressure:

$$(N)_{60} = C_N \times N_{60}$$

$$C_N = 9.79\,(1/\sigma_v')^{1/2}$$

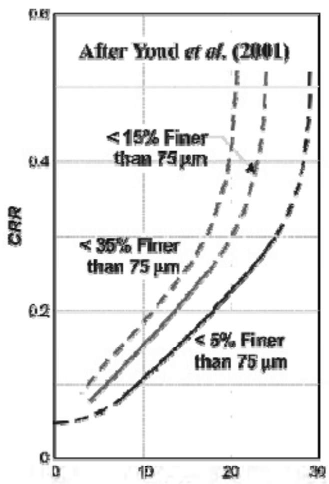

Figure F-2 (for SPT data)

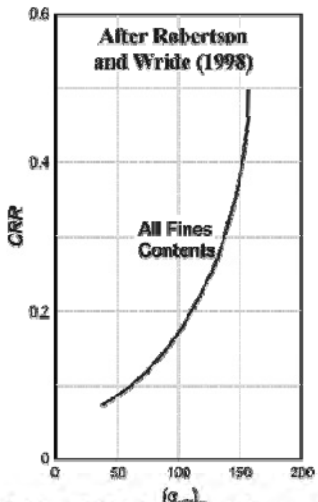

Figure F-2 (for CPT data: in "factor of safety" calculation in column 2 of page 24 this figure is wrongly cited as F-6)

Figure F-4 provides a plot for k_m. Algebraically, the relationship is simply $k_m = 10^{2.24} / M_w^{2.56}$ subjected to $k_m \geq 0.75$

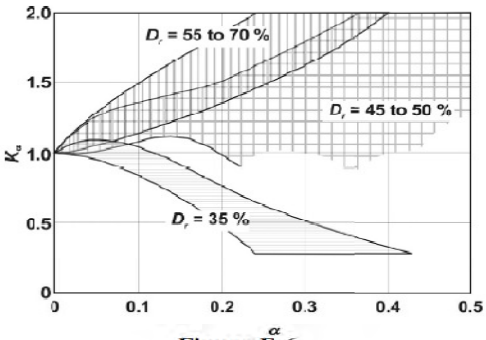

Figure F-6

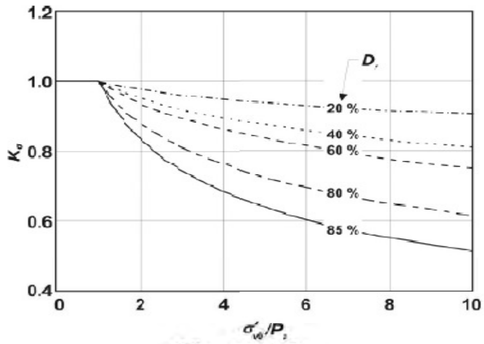

Figure F-5

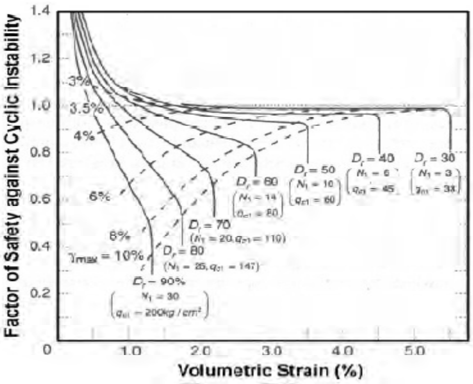

Figure F-8

$C_N = 9.79\,(1/169.7)^{1/2} = 0.75$

$(N)_{60} = 0.75 \times 17 = 13$

Critical stress ratio resisting liquefaction:

For $(N)_{60} = 13$, fines content of 8%

$CSR_{7.5} = 0.14$ (Figure F-2)

Corrected Critical Stress Ratio Resisting Liquefaction:

$CSR_L = CSR_{7.5}\; k_m\; k_\alpha\; k_\sigma$

k_m = Correction factor for earthquake magnitude other than 7.5 (Figure F-4)

$= 1.00$ for $M_w = 7.5$

k_α = Correction factor for initial driving static shear

(Figure F-6)

= 1.00, since no initial static shear

k_σ = Correction factor for stress level larger than 96 kPa (Figure F-5)

= 0.88

$CSR_L = 0.14 \times 1 \times 1 \times 0.88 = 0.12$

Factor of safety against liquefaction:

$FS_L = CSR_L / CSR_{eq} = 0.12 / 0.18 = 0.67$

Percentage volumetric strain (%ε)

For $CSR_{eql} = CSR_{eq} / (k_m\; k_\alpha\; k_\sigma)$

= 0.18 / (1x1x0.88) = 0.21

$(N_1)_{60} = 13$

%ε = 2.10 (from Figure F-8)

Liquefaction induced vertical settlement (ΔV):

(ΔV) = volumetric strain x thickness of liquefiable level

$= 2.1 \times 3.0 / 100 = 0.063m = 63mm$

Summary:

Analysis shows that the strata between depths 6m and 19.5m are liable to liquefy. The maximum settlement of the soil due to liquefaction is estimated as 315mm (Table 10.2)

Table 10.2: Liquefaction Analysis: Water Level 6.00 m below GL (Units: Tons and Meters)

Depth	%Fine	σ_v (kPa)	σ_v' (kPa)	N_{60}	C_N	$(N)_{60}$	r_d	CSR_{eq}	CSR_{eql}	$CSR_{7.5}$	CSR_L	FS_L	$\%\varepsilon$	ΔV
0.75	11.00	13.9	13.9	9.00	2.00	18	0.99	0.15	0.14	0.22	0.25	1.67	-	-
3.75	16.00	69.4	69.4	17.00	1.18	20	0.94	0.15	0.14	0.32	0.34	2.27	-	-
6.75	12.00	124.9	117.5	13.00	0.90	12	0.90	0.15	0.15	0.13	0.13	0.86	2.30	0.069
9.75	8.00	180.4	143.6	18.00	0.82	15	0.85	0.17	0.18	0.16	0.15	0.88	1.90	0.057
12.75	8.00	235.9	169.7	17.00	0.75	13	0.81	0.18	0.20	0.14	0.12	0.67	2.10	0.063
15.75	7.00	291.4	195.8	15.00	0.70	10	0.76	0.18	0.21	0.11	0.09	0.50	2.50	0.075
18.75	6.00	346.9	221.9	26.00	0.66	17	0.72	0.18	0.22	0.18	0.15	0.83	1.70	0.051
Total Δ														**0.315**