"Stronger Is Not Necessarily Better" - The Significance of Tests and Properties of Civil Engineering Materials

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ABSTRACT: Oftentimes, the Consulting Engineer is confronted with Materials Test Results from which he has to make judgments that would have a potentially large impact on the Project's cost, schedule, quality or safety. The decisions made depend to a large extent on the Engineer's knowledge and familiarity with the test procedures, test limitations, significance of the test parameters as it affects his design, the acceptance criteria and material behavior under load or in differing environments. In the strength testing of rebars for example, higher yield stresses during test do not necessarily mean **better** as other test information/parameters need to be evaluated or before acceptance or conclusions could be made. In the testing of concrete, several failures in a batch of cylinders do not necessarily mean that the batch should be condemned as the statistics need to be evaluated before such a drastic action is even contemplated. In the compaction of soils, excessive compaction leads to breakdown and degradation of the Soil Fabric contrary to ordinary laymen's expectations *"That the more you pound, the harder the ground."* The Engineer should therefore be equipped with adequate knowledge and understanding of the test procedures material properties and material behavior in order to make intelligent and *"Informed"* judgment calls Engineering Judgment in its truest sense. This paper hopes to open the way to a greater understanding of this important aspect of our day to day practice of the profession in the real world.

1.0 INTRODUCTION

In the Building Industry, *"Stronger"* has always been synonymous to *"Better"*. This has been manifested in, and reinforced by, common beliefs due to the survival of archeological structures which because they were Built "Strong", have actually survived. However, most of these ancient structures have survived through sheer massiveness and more than liberal use of materials such as masonry blocks and mortar. Nevertheless, not even all of these have survived the ravages of Earthquakes in our country. Even those which have survived show scars or damage due to Earthquakes.

It is disheartening to note that this mistaken belief has creeped into our present day practice and most of the time *Stronger, Harder, Bigger, Stiffer, etc.,* have always been *Better !*

Alas, present day knowledge of material behavior and performance as borne out by Laboratory Tests simulating actual service loading under Earthquake or other conditions have shown that **Stronger is not always Better !**

This paper hopes to highlight some fallacies in the Design of structures, be it Buildings, Roads or Dams which tended to overdesign or increased strengths by choice or by accident.

In some instances, as we shall find out later on in this paper, higher strengths could lead to bigger problems and may surprisingly at times trigger an earlier failure in our structure than if the structure were purposely made *"Weaker"*.

Of significance in this discussion is the appreciation of the Test Parameters and results of Laboratory Tests on Civil

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Engineering Materials. A clearer understanding of the test results allows the Designer and Consulting Engineer to render proper judgment calls and Engineered Decisions that are supported by the material's characteristic behavior and the limits imposed by Code or by Standards of Practice and its behavior under loads.

We shall discuss these very important considerations and the role that testing plays for:

- **Reinforcing Bars**
- **Concrete**
- **Structural Steel**
- **Soils**

Sometimes, Design Engineers accept materials substitution without understanding its characteristic behavior. This could lead to objectionable or dangerous consequences.

This is what we are going to find out in order to support the statement **"Stronger is not necessarily better".**

First, let us try to understand what happens to Reinforced Concrete under **severe** Seismic Loading conditions.

In order to do so, we have to go to our Basic Fundamentals of Concrete Design which we summarize:

1. In Reinforced Concrete design, we were taught that it is desirable to attain a reinforcement ratio that is below $\mathbf{P_b}$ or balanced reinforcement **(0.75 $\mathbf{P_b}$).** This is to ensure that yielding or failure is initiated first on the steel ahead of the concrete compression block to avoid an explosive, Brittle and sudden failure.

2. Under seismic loading, ductility plays a very important role for both concrete and steel structures. Ductility is critically important in RC Structures under **Zone 4** for several reasons:

    - We would like to prevent sudden failures or collapse without warning.

    - Initial yielding or plastic hinging has to be initiated at an early stage under severe seismic loads to allow dissipation of energy. Failure to do so build ups larger inertia forces that need to be absorbed by the structure thus causing more severe damage or sudden collapse.

    - Design under severe Earthquake loadings requires that total collapse does not ensue although the serviceability may be impaired to a point beyond practical repair.

This brings us to focus on just what is necessarily needed to satisfy the foregoing fundamental requirements.

Very critical to this is that yielding is initiated at an early stage where it was assumed by the Designer in compliance with the code for Seismic Design. This can only happen if the **Yield Stress** is low - *(Not Stronger!)* but still satisfying the design Yield Stress (YS) specified in the Code and used by the Designer.

Why is this important **?**

- Postponing the yield as we have said **increases** inertia forces which the structure has to absorb.

- But more important, if yield does not occur at the design yield, Bond and Shear stresses reach critical levels earlier, thus initiating a sudden failure in the structure by Brittle Behavior.

- If in addition to this, the Yield Stress approaches the Ultimate Tensile Stress very closely *(reduced yield region).* Gradual formation of Plastic Hinges is aborted and ultimate strength is reached causing sudden collapse.

- A measure of how far apart is the **Yield Stress** and the **Tensile Stress -** the **TS/YS** ratio, is a measure of the ability of the structure to undergo inelastic rotation and absorb energy and dissipate it by deformation or yielding. As the structure loses its stiffness in response to a strong ground motion, its capability to dissipate energy increases. These tend to reduce the response acceleration or lateral inertia forces that develop during deformation of the structure.

The ACI Code & PNS 49 both call for a **TS/YS** ratio of **1.25.**

This sets the minimum distance between the yield and tensile stresses for obvious reasons - To allow sufficient time to develop plastic *rotation and promote energy dissipation* before collapse is induced.

Implicit in all these is there is a need to impose a ceiling on the yield stress to the level assumed in the design. Thus, it is a fallacy and highly erroneous to accept higher yield strength rebars because they are **stronger,** in total disregard of the design assumptions !

"Because Stronger is not necessarily better!"

As a specific example, sometimes we encounter situations where the Engineer blindly accepts substitution of Higher Grade Rebars *(As when he specifies grade 40 and then accepts substitution by grade 60 without any qualifications)* than what he or she used in the design in total ignorance of the need to limit the yield stress.

To compound this, unscrupulous suppliers try to pass on **Non Standard** rebars or Non Approved Rebars that have significantly very high yield stresses very much closer to the Tensile Stress *(A reduced yield region),* a TS/YS ratio approaching unity. Therefore inelastic rotation is relatively short and failure ultimately ensues.

Therefore, we can conclude that **"Stronger is not necessarily better".**

In another vein, very high tensile *(and yield stress)* lead to Brittle Failure mode as the materials really are brittle. But this is another story.

In addition to controlling rebar strengths, the reinforcement ratio $\mathbf{P_b}$ should also be controlled both ways.

In seismic design, the two extremes are critical. It is necessary that the reinforcement ratio be controlled:

- By setting **minimum** limits to the reinforcement ratio **200** $\mathbf{b_w d/fy}$

- By setting **maximum** limits = **0.75** $\mathbf{P_b}$**.**

There are compelling reasons for the above requirements.

It would not be advisable to severely **under reinforce** the RC Structural element because the **cracking moment Mc** would be reached first rather than the yield moment.

In the first instance, a single crack development would cause a sudden catastrophic collapse because gradual yielding and straining is not possible.

In the other extreme, **over reinforcement** beyond the balanced reinforcement requirement **"**$\mathbf{P_b}$**"** initiates early overstress in the concrete compression block rather than allowing gradual yielding accompanied in the rebar by gradual deflections which provides ample warning to the occupants unlike a compression type failure which is essentially **explosive** and sudden.

Therefore, **"Stronger is not better"**

Lest we are lulled into generalizations, we also add another admonition:

"Less *(weaker)* is also not better"

ACI 318 thus imposes the following restrictions for seismic design in .R.C as follows:

Requirement		*ACI 318*
	Longitudinal Reinforcement in Beams	
200 b_w d/fy	**minimum** reinforcement ratio to avoid initiation of cracking moment.	**21.3.2.1**
0.75 P_b	**maximum** reinforcement ratio to ensure initiation of yielding in the steel and avoid an explosive type failure.	**10.3.3**
0.025	**maximum** reinforcement ratio **r**	**21.3.2.1**
	Column Reinforcement Ratios	
0.01 minimum	**21.4.3.2**	
0.06 maximum		

2.0 REBARS

Tests on rebars is guided by Philippine National Standards (PNS) PNS-49:1991 *"Steel Bars for Concrete Reinforcement - Specification"* by the Bureau of Product Standards covering the following grades of steel rebars:

Grade	230	For both Weldable
	275	and non weldable

415 hot rolled steel rebars

Looking at the table, it is interesting to note that the requirement for TS/YS ratio *(As indicated by **)* only **applies** to Grade 415 Weldable Steel whereas ACI 318 and its commentary is very explicit that the **TS/YS** ratio should be **1.25** without exception or qualification as to rebar type for Seismic Design.

Thus, there is a real need **to amend** PNS 49-1991, to amend the Table so as to cover all Rebar Grades and Types *(Weldable and Non Weldable to ensure adequate performance in a High Seismic risk location)* and thus comply with the ACI Code and the NSCP.

Table 2 - Mechanical Properties

Class	Grade	Yield Streng th MPa	Tensile Strengt h MPa	Specime	Elonga - tion in 200m m,	Bendin g angle,	Diameter of Pin (d = nominal diameter
Hot-Rolled Non-Weldable Deformed Steel Bar	230	230	390	D < 25 D ≥ 25	18 16	180	3d 4d
	275	275	480	D < 25 D ≥ 25	10 8	180	4d 5d
	415	415	620	D < 25 D ≥ 25	8 7	180	5d 6d
Hot-Rolled Weldable Deformed or Plain	230	230	390	D < 25 D ≥ 25	20 18	180	3d 4d
	275	275	480	D < 25 D ≥ 25	16 14	180	4d 5d
	415	415*	550*	D < 25 D ≥ 25	14 12	180	5d 6d

* ***Yield strength maximum of weldable deformed or plain steel bar = 540 MPa***

‡ ** ***Tensile strength shall not be less than 1.25 times the actual yield strength.***

\+ ***Plain steel bars are only available in grade 230. Other grades are subject to buyer's and manufacturer's agreement.***

This clearly has to be amended, because no less than the ACI Code & the NSCP call for a **TS/YS** ratio of **1.25** without any exclusion for rebars used in regions with high seismic risk

This brings us to the significance of the **TS/YS ratio.**

ACI 318 explicitly calls for a **TS/YS** ratio of **1.25** without exception, for highly seismic Zone S *(Zone 4).* It also stipulates several important requirements as follows:

1. The specified yield strength **YS** should not be exceeded by more than *18,000 psi (124 MPa).*

2. The **TS/YS** ratio shall not exceed **1.25**

PNS 49 is also explicit in that it specifies a **minimum** and **maximum** permissible stress for yield stress.

What the two foregoing requirements clearly state is that a limit has been set on the yield stress (YS).

Why is this so?

This is clearly explained in ACI 318 Subsection 21.2.5 which we quote as follows:

21.2.5 - Reinforcement for Members resisting Earthquakes

"Reinforcement resisting earthquake induced Flexural Stresses and axial forces in frame members and in wall boundary elements shall comply with ASTM A-706, ASTM A-615, Grade 40 & Grade 60 reinforcement shall be permitted in these members if:

a) *The actual yield strength based on mill tests does **not** exceed the specified yield strength by more than 18,000 psi (124.1 MPa).*

b) *The ratio of the actual ultimate tensile strength to the actual tensile yield strength is not less than **1.25.***

Code Commentary R 21.2.5

"Use of longitudinal reinforcement with strength substantially higher than assumed will lead to higher shear and bond stresses at the time of development of yield moments. These conditions may lead to brittle failures in shear or bond and should be avoided even if such failures may occur at higher loads than those anticipated in design. Therefore, a ceiling is placed on the actual yield strength of Steel.

The requirement for an Ultimate Tensile Strength larger than the yield strength of the reinforcement is based on the assumption that the capability of a structural member to develop inelastic rotation capacity is a function of the length of the yield region along the axis of the member. In interpreting experimental results, length of yield region has been related to the relative magnitudes of ultimate and yield moments. According to that interpretation, the larger the ratio of ultimate to yield moment, the longer the yield region."

Members with reinforcement not satisfying that condition can also develop inelastic rotation, but their behavior is sufficiently different to exclude them from direct consideration on the basis of rules derived from experience with members reinforced with strain hardening steel."

3.0 CONCRETE

Oftentimes, the Structural Designer specifies the Design Strength **(f'c)** for his RC Design and leaves it at that. However, when reports of cylinder tests come in and there are reported failures, he responds immediately by ordering concrete cores to be extracted or worse, a load test. Both responses are costly and often not necessary!

All that is probably initially required is a complete understanding of the possible variabilities that can occur in concrete and also how as designer/specifier, he can control these variabilities to desirable limits and thus have a firm basis for acceptance/rejection.

What often happens is that the Designer treats concrete test specifications as fixed and any failure as absolute failures.

Concrete, as we have said, is a highly variable material and as such is subject to the laws of statistics. When the Designer specifies a Design Strength **(f'c),** he in effect should be requiring something higher than this value in order to ensure that failures are within acceptable limits. Implicit in this statement is the need to specify a *Required Average Strength* **f'cr** that is greater than **f'c.**

When the Engineer unrealistically refuses to accept the variability in concrete strengths and consistently demands that **no tests** fall below the Specified Design Strength **(f'c),** he unreasonably inreases the cost of the project, as the supplier has to increase his required strength design to ensure that his breaks do not fall below **f'c.** Thus, in effect, but perhaps without knowing it, the Designer imposes **higher** strength concrete which he does not need and forces the supplier to provide *overly conservative Mix Designs.*

Since stronger concrete is definitely more expensive than an *Engineered* concrete specification, the theme **"Stronger is not necessarily Better** again applies.

When the Designer/Specifier expects that concrete compression test results to be always equal to or greater than the Specified Concrete Strength **(fc')** he unwittingly causes problems other than increasing the cost of concrete.

Higher strengths are obtained by limiting the Water Cement (WCR) Ratio (which causes a retrograde effect on the workability of the product) or by higher cement content.

However, the foregoing could cause some other problems:

- Use of water reducing admixtures or plasticizers to increase workability means added cost per cubic meter.

- Increases in cement content brings attendant problems of higher heat of hydration generated which could cause thermal cracking or high shrinkage cracking.

Thus, it would be necessary to gain a fundamental understanding of the variability of concrete and to accept the possibility that failures can and do occur even in a well supervised concrete batching, sampling and test operation or system. What is more important is to know how to control these variables, so that they can be placed *within limits of acceptability* in consideration of the criticality or demands of the structure. Evidently, not all structures require or should impose very strict demands on strength since in some structures, durability considerations are more important.

In a parallel vein, a nuclear containment structure would definitely have more critical demands on quality and strength as say an irrigation canal.

Thus, the use and application of statistical procedures as recommended by **ACI 214 "Recommended Practice for Evaluation of Strength Test Results of Concrete"** is critically important.

ACI 214.3R Approximately describes the need to apply Statistical Procedures in specifying strengths for concrete:

> ***"Specifying the Strength of Concrete***
>
> *When the Structural Engineer specified a "Design Strength" for his structure he in effect specifies a Specified Strength* ***(f 'c).***
>
> *Since the strength of concrete follows the* ***Normal*** *distribution curve, if the average strength of the concrete is approximately equal to the specified strength, one half of the concrete will have a strength less than the specified strength. Because it is usually not acceptable to have one half of the strength tests lower than specified stength, the average strength must be higher than the specified strength by some factor.*
>
> *The specification writer, in consultation with the Engineer, writes a specified strength and the percentage of low tests that are considered acceptable for that class of concrete.* ***ACI 318,*** *"Building Code Requirements for Concrete" provides guidelines for selecting acceptable number of low tests."*

An example of a statement for strength in the specification might read:

"The average of all Strength Tests shall be such that not more than one (1) test in Ten (10) shall fall below the Specified Strength ***fc'*** *of 3,500 psi"*

In turn, the concrete producer, in order to meet the above specifications would have to provide a strength that is definitely higher than **f 'c,** called the required Average Strength **(f 'cr).** The Required Average Strength can be determined from the following formula:

$$f'cr = f'c + pS$$

where:

f 'cr	=	required average strength
f 'c	=	specified strength or design strength
p	=	probability factor based on the percentage of tests the designer will allow to fall below f 'c
S	=	expected Standard Deviation for the project

Use of the **Normal** distribution curve to obtain the required average strength is illustrated in Fig. 3.1.

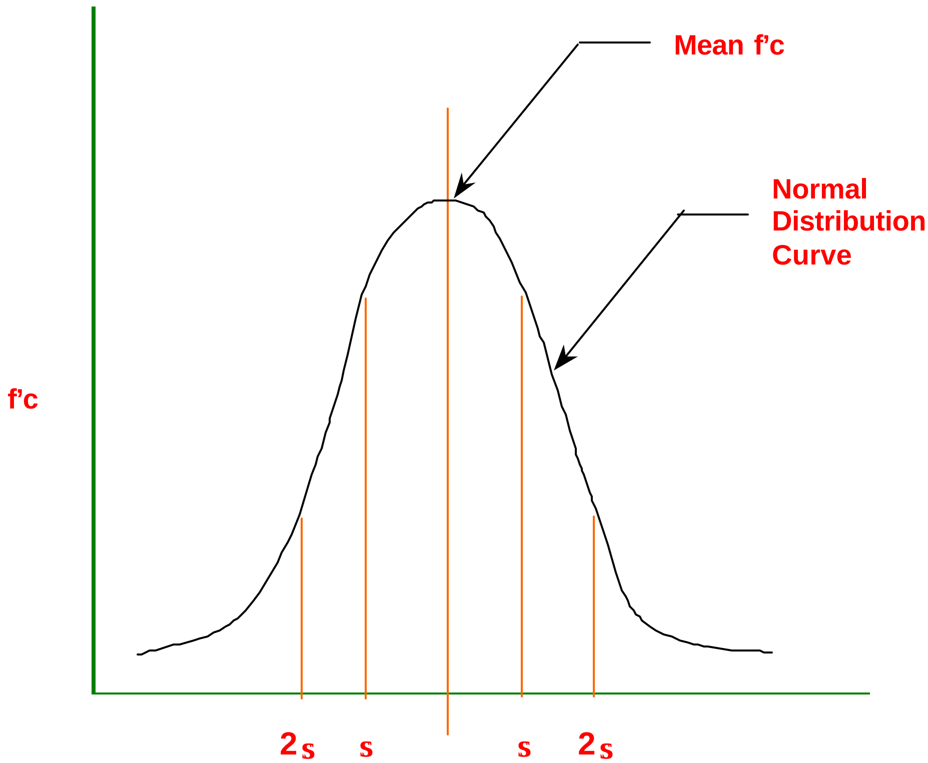

Fig. 3.1 - Normal Distribution Curve

To calculate the required average strength, the Engineer must decide the specified strength and what percentage of tests falling below the specified strength will be allowed. When the decision has been made on an acceptable percentage of low tests, the probability factor can be determined using the properties of the Normal distribution curve. The probability factors for various percentages of low tests are given in Table 2 below:

Required average strength f'c	Percentage of low tests	Required average strength f'c	Percentage of low tests
f'c + 0.00s	50.00	f'c + 1.60s	5.50
f'c + 0.10s	46.00	f'c + 1.70s	4.50
f'c + 0.20s	42.10	f'c + 1.80s	3.60
f'c + 0.30s	38.20	f'c + 1.90s	2.90
f'c + 0.40s	34.50	f'c + 2.00s	2.30
f'c + 0.50s	30.90	f'c + 2.10s	1.80
f'c + 0.60s	27.40	f'c + 2.20s	1.40
f'c + 0.70s	24.20	f'c + 2.30s	1.10
f'c + 0.80s	21.20	f'c + 2.40s	0.80
f'c + 0.90s	18.20	f'c + 2.50s	0.60
f'c + 1.00s	15.90	f'c + 2.60s	0.45
f'c + 1.10s	13.60	f'c + 2.70s	0.35
f'c + 1.20s	11.50	f'c + 2.80s	0.25
f'c + 1.30s	9.70	f'c + 2.90s	0.19
f'c + 1.40s	8.10	f'c + 3.00s	0.13
f'c + 1.50s	6.70		

The Standard Deviation **S** is obtained by analyzing the Concrete Producer's data. Since the Standard Deviation for a project is not known at the beginning of a project, Chapter 4 of ACI 318 permits the substitution of a Standard Deviation calculated from at least 30 consecutive strengths on concrete produced at the proposed concrete plant using similar materials and conditions.

ACI 318 is more specific in the selection of the Required Average Strength **f'cr** to be used in the proportioning of concrete mixes.

ACI 5.3.2.1 States:

*"Required Average Compressive Strength f'cr use as the basis for selection of concrete proportions shall be the larger of Eq (5-1) or (5-2) using a Standard Deviation **S** calculated in accordance with 5.3.1.1 or 5.3.1.2*

$$f'_{cr} = f'_c + 1.34\,S \qquad (5\text{-}1)$$

$$f'_{cr} = f'_c + 2.33\,S - 500 \qquad (5\text{-}2)$$

ACI 5.3.2.2

When a concrete production facility does not have field strength test records for calculation of Standard Deviation meeting requirements of 5.3.1.1 or 5.3.1.2, Required Average Strength fc'r shall be determined from Tabl3 5.3.2.2 and documentation of average strength shall be in accordance with requirements of 5.3.3."

TABLE 5.3.2.2 - Required Average Compressive Strength when data are not available to establish a

Standard Deviation.

Specified f'c (psi)	Required Average Compressive Strength f'cr (psi)
> 3000 psi	f'c + 1000 psi
3000 to 5000 psi	f'c + 1200 psi
> 5000 psi	f'c + 1400 psi

Thus, from the foregoing, it can be clearly seen that there is a rational way of specifying concrete strength which would relatively be more economical than an arbitrary and ambiguous requirement that absolutely **No** tests fall below the specified f 'c. This implicitly means that the Designer is in effect specifying "*stronger*" concrete. Implicit with our understanding is the acceptance of failures within the batch but which are within acceptable limits on the number of failures. The Design Engineer therefore needs to have a more thorough appreciation and knowledge of the variable nature of concrete as a Civil Engineering Material and how he can control it through proper application of statistical methods not just by specifying "*stronger*" concrete.

Evaluation and Acceptance of Concrete

Knowing what to specify and what to expect in terms of the variability of concrete test results is only half of the picture.

Having a clear basis for acceptance/rejection is the other half.

This brings us to just exactly what is meant by a "Test".

A test is defined in 5.6.1.4 of ACI 318 as follows:

> *"A strength test shall be the average of the strengths of* ***two*** *cylinders made from the same sample of concrete and tested at 28 days or at the test age designated for determination of f'c."*

A lot of times, Design Engineers or even Project Engineers reject a concrete batch on the basis of a **single** cylinder break and without due consideration of the established criteria for Aceptance/Rejection which are stated below:

"5.6.2.3 Strength level of an individual class of concrete shall be considered satisfactory if ***both*** *of the following requirements are met:*

a) Every arithmetic average of any three consecutive strength tests equals or exceeds f'c.

b) No individual strength test (Average of Two Cylinders) falls below f'c by more than 500 psi."

4.0 STRUCTURAL STEEL

Similar considerations govern the use of structural steel in highly seismic regions such as what we are in.

Particularly for built up sections which are commonly used in the Philippines, Substandard Plates from some Eastern European Mills are passed on as **A-36 Steel.**

When tested, the steels exhibit very high yield stresses *(High Carbon Content)* and Tensile strengths just slightly above the yield stress.

In effect, these steels would exhibit Brittle or non ductile behavior during an earthquake. Thus collapse would also probably be sudden.

Why do these proliferate in the Philippines?

It is because of several things:

- Lack of knowledge of the Brittle Behavior of the steel.
- Unscrupulous suppliers who try to pass this on as A-36 or other acceptable specified grade steel
- Plain ignorance on the part of the Designer, Specifier or Project Manager
- Tests have not been performed.

What is worse, if these are used with welded connections, serious incompatibility with the welding procedures specified for say A-36 steel and the high YS and TS steels could produce defective connections and **embrittlement** in the joints.

Again, sudden failure on the joints could result.

Often, the Design Engineer accepts these test results blindly since the steel strength test results are **"Stronger"** than was specified and therefore necessarily **"Better".**

This is very much farther from the truth as the theme in this paper aptly applies:

"Stronger is not necessarily Better"

Although not within the scope of this paper, and since this has been extensively discussed in other fora, joints in structural steel complying with the current AISC and local codes have exhibited failures during the **Loma Prieta** Earthquake.

What does this tell us?

Making the joints compliant with what was then an existing code or even stronger does not guarantee proper structural performance.

5.0 SOILS

Soil is the ultimate structure on earth because all man made structures eventually rest on the soil. Extensive in occurrence, man has to contend with a highly variable material.

However, to some extentman can control the Quality of Soil through stabilization, amelioration or ground improvement and this is where the problem lies.

Always, the target is to produce a stronger material either by overcompaction or by stabilization.

Most of the time this is done in total ignorance of soil mechanics principles and soil behavior.

Compaction

The situation is best illustrated in the most extensively used procedure in Civil Works: - compaction

Compaction in granular soils is sometimes carried to extremes - Heavier compactors, - numerous passes more than what is required - just to make sure that what is attained is *"stronger"*.

Little do people know that overcompaction is not beneficial and in fact degrades the density initially obtained.

In **clean granular soils** subjected to overcompaction, the soils shears and density collapses beyond the optimal compactive energy *(number of passes).*

In **cohesive soils**, overcompaction results in remolding sensitive clays and causes strength loss.

In **expansive** or highly swelling soils, the compactive effort needs to be reduced and the Moisture Content kep wet of optimum to reduce swell potential and heaving.

In a former engagement, the author's attention was called, as the results of Field Density Tests were being questioned vehemently by the contractor because the clean granular soils have consistently failed to meet specifications.

Upon investigation, the following observations were made:

1. The sub contractor was delivering from **9** to **14** passes on the soil with 15 MT vibratory compactors because:

    1.1 Field Density Tests showed low densities consistently below the Target MDD after each day's test.
    1.2 Fuel is provided free by the Prime Contractor
    1.3 Equipment is paid based on operating hours.

2. The soils are being compacted at *"Optimum Moisture Content" (OMC)*

3. Parallel Cracks transverse to the direction of compactor travel have formed in the overcompacted soils.

Trial compaction works were ordered and it was found out that only 3 to 4 passes were needed to reach specified densities.

In addition, it was finally made clear, but with much difficulty, that there is no **"Optimum Moisture Content"** when applied to clean granular soils. The Contractor all the while was taking pains in controlling moisture to **OMC** due to a lack of understanding of the behavior of clean granular soils subjected to compaction. The soil either has to be very very dry or very very wet before compaction to attain maximum density.

As a result, the subcontractor suffered a severe reduction in rental revenues as the compactor fleet was reduced by more than 50% although additional water tankers were needed.

The Prime Contractor in turn made substantial savings and the construction schedule was substantially speeded up.

Again this is another case where

"*Stronger* is not necessarily Better"

Why is OMC not relevant in Clean Granular Soils?

Let us look at the characteristic compaction curves for Granular Soils.

Looking at the compaction curve above for clean granular soils, immediately tells us that this is very much different from the normal *parabolic* shape of fine grained and cohesive soils. The twin peaks P1 & P2 indicate that the soil can either be compacted *very very* ***dry*** or *very very* ***wet*** and that OMC is not relevant

Instead of a Parabolic Shape, the **"S"** Curve can be clearly seen. The *"Trough"* between 0 to 12% MC *(varies with soil type)* is the bulking moisture content where surface tension of the moisture holds the grains apart. Thus density is low.

Without understanding the characteristic behavior of the soil in the Moisture Density Curve, very costly and highly erroneous compaction procedures would result.

Case where Building "Strengthening" caused more Distress

Another extreme reaction due to a wrong perception of Soil Behavior and its telltale effects happened in one project where we were involved to evaluate a Building that was **"Sinking".**

Earlier remedial measures directed towards the mistaken assumption that *settlement* was occuring resulted in a costly but unneeded measure, but worse, it even aggravated the Structural Damage to the Building.

What was then wrongly perceived as *"settlement"* was in fact **heaving.** Since the perception of the direction of movement was based on guesswork, and because no tests were performed, incorrect or inappropriate remedial measures were implemented as follows:

Phase 1	**-**	Extensive Structural Repairs and **Strengthening** were made on the Waffle Slab RC Deck.
Result	*:*	Distress continued despite the repairs.
Phase 2	**-**	Because Phase 1 continued to "settle", Piled Foundations were specified on the next Phase. In effect making the foundation ***stronger*** by using Piles to bypass ***"weak"*** soils.
Result	*:*	The Piled foundations, rather than reducing the damage, caused more severe damage in a shorter period of time than the level of damage sustained by Phase 1 !

The two Buildings were badly cracked but as stated, the Pile Supported Buildings cracked **more extensively** and **more severely** than the unpiled structure !

Subsequently, during our investigation, we found out that **heaving** and not settlement was occurring. The only solution to arrest the cracking was to remove a layer of Base Course Material composed of slag from a Steel Mill.[3]

[3] **Morales, E.M. *"Structural and Functional Distress due to Slag Expanion"* International Conference on Case Histories In Geotechnical Engineering". June 1-4, 1993, St. Louis, Missouri, USA.**

The Contractor and Owner's Project Engineer thought that slag being **heavier** and

stronger would be better and cheaper than normal granular Base Course since it was available for the asking and compacts well.

Unfortunately, the slag corroded under the very acidic ground water resulting in expansion and heaving.

"The ***"Stronger"*** Slag Base Course proved to be not only inferior but also caused heavy damage than a weaker material - in this case granular Base Course

Going back to the Piles, which were perceived to offer a **"Stronger"** Foundation, severe cracking and more extensive damage resulted from the restraint offered by the Piles which served as Anchor Piles preventing the Phase 2 Structure from rising.

The restraint on the Walls and Columns caused more severe cracking than the Phase 1 Building since the earlier building was founded on spread footings and was relatively more free to ride the heaving than a *"Stronger"* Pile *"Supported"* (Restrained) Structure.

In these two instances involving the R&D facility, wrong understanding of the soil behavior and lack of any tests done on the soils resulted in a very expensive but ineffective and far more damaging response to the problem.

Critical Engineering Judgment was definitely needed in this case.

6.0 CONCLUSION

The foregoing discussions and examples based on real world experiences show that sometimes wrong reactions or responses to the problem bring about unwanted consequences.

Particularly in the practice of our profession, the normal tendency when a problem occurs is to **strengthen** or use **stronger** materials to ensure an "imagined" *factor of safety* which in reality is a double bladed *factor of ignorance !*

We can not allow this to happen as this can cause unwanted and oftentimes dangerous outcomes.

We should strive to understand material behavior and the Environmental Influences which can alter or totally change the performance that we expect from our structures.

It is necessary for all of us gathered here to spread this message to our subordinates and apprentices so that the lessons of the past will not be repeated.