The Mysterious 1/3 Stress Increase

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It has been customary throughout three or four generations of structural engineers to use a higher allowable stress when considering the stresses produced by wind in a structure. Much later, the same provision was allowed for earthquake-related stresses. Recently, this increase has come under attack from many quarters and there has been some confusion as to what was the rationale for permitting this increase in the first place. Just what physical phenomenon is it supposed to account for? When was it first introduced and why? If it was valid at the time of its origin, is it still valid today?

The criticism has come from various sources. For example, engineers on the west coast have been lobbying to get rid of the provision for several years, primarily for earthquakes, but wind usually gets caught up in the fervor. Each year some building official proposes to the Uniform Building Code that the increase be abolished for both earthquake and wind. Each year the proposal is defeated. Since the earthquake provision came later and was probably borrowed from the wind provision, let us remove earthquakes from consideration here and investigate the validity of the permitted stress increase for *wind only*.

At a Workshop on Wind Loads at Northwestern University in June 1976,[1] it was pretty clear that there was much confusion over the $1/3$ stress increase and exactly what types of conditions it is supposed to account for. Some members of the ANSI A58.1 Wind Load Committee expressed the belief that it represents a low probability that maximum live load and maximum wind load would ever occur simultaneously. This is the "probability factor" of 0.75 which is to be multiplied by loads in combination with wind, which appears on page 10 of the A58.1 Standard.[2] It goes on to say that:

> "An increase in the allowable stresses will not be allowed in conjunction with a decrease due to the above load combinations."

Thus, the traditional $1/3$ stress increase is *not* permitted if one has already multiplied the loads by 0.75. This is reasonable *if, in fact, the $1/3$ increase is a probability factor.*

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But suppose it is not. Suppose it is to account for phenomena which have nothing to do with the probability of loads acting in combination?

The confusion is aided by the fact that multiplying the allowable stresses by $4/3$ accomplishes the same thing as multiplying the loads by $3/4$.

It is natural to assume, then, since $3/4$ is the inverse of $4/3$, that one is simply being transferred to the other side of the equation, and to put them *both* in would account for the same effects twice.

It has been my contention that, although it appears impossible from a mathematician's viewpoint, $3/4$ is *not* the inverse of $4/3$! That is to say, the traditional increase in allowable stress for wind has *no relation* to the probability that maximum live load and a maximum wind load will occur simultaneously.

This controversy motivated me to conduct my own informal, unfunded research project to see what I could uncover on the subject. I have asked many prominent steel designers from diverse geographic locations in the past year, if they use the $1/3$ stress increase and what they thought it was for. A great many simply said that AISC permits it, so they use it, but don't know why. Of the ones who offered a possible rationale for its use, their replies can be fitted into three categories:

1. The action of wind on a structure is highly localized and of very short duration. Therefore, it is not necessary to have as high a safety factor when designing for wind loads.

2. The properties of some materials change with the rate of loading. Steel, when loaded rapidly, will show a higher yield strength than when it is loaded slowly. The $1/3$ stress increase merely reflects the increase in properties due to rapid loading and does not diminish the safety factor.

3. The $1/3$ stress increase reflects the low probability of maximum live and wind loads occurring simultaneously. Therefore, when checking $D + W$ only, it should not be used.

The next step was a literature search to see which of these opinions, if any, could be supported. The oldest reference I could find to using an increased allowable stress for wind was in Cooper's railway bridge specifications of 1896.[3] This permitted a 1/4 increase, but gave no reason. A. J. DuBois[4] recommended, also in 1896, an allowable stress for beams of 10,000 psi and an allowable for lateral bracing of 15,000 psi. Although no statement is made regarding the reason for a higher allowable stress for bracing, it could be interpreted as a 50% increase because of wind forces.

Ketchum in his specifications for mill buildings[5] of 1903, says:

> "When combined direct and flexural stress due to wind is considered, add 25 per cent to the above allowable tensile and compressive stresses."

Like Cooper, he does not offer any explanation as to *why* this should be permitted.

The New York City Building Code[6] of 1904 had this statement:

> "In calculations for wind bracing the working stresses set forth in this code may be increased by 50%."

Again, no reason is given.

In 1923, the American Institute of Steel Construction published its first specification.[7] In it are these words:

> "*Combined Stresses*—For combined stresses due to wind and other loads, the permissible working stress may be increased 33 1/3 per cent, provided the section thus found is not less than that required by the dead and live loads alone."

> "*Members Carrying Wind Only*—For members carrying wind stresses only, the permissible wind stresses may be increased 33 1/3 per cent."

This provision in the AISC Specification remained virtually unchanged for 51 years, except for the inclusion of seismic stresses. In 1974, Supplement #3 added a qualifying paragraph:

> "and provided that stresses are not otherwise* required to be calculated on the basis of reduction factors applied to design loads in combinations"

The footnote says: "*For example, see ANSI A58.1, Section 4.2."

In 1930, Robins Fleming wrote a comprehensive text on wind stresses in buildings,[8] and put forth the first logical explanation of the mysterious 1/3 increase that I have been able to find:

> "*Because wind loads are intermittent and seldom reach their maximum,* greater working stresses are permissible for them than for live and dead loads. This is recognized generally by engineers and has found a place in most building codes. In the New York City Code, where at present (1929) a working stress of 16,000 psi is specified for tension in rolled steel, an excess of 50 per cent of stresses prescribed elsewhere in the code is allowed for combined wind, dead, and live loads, provided that the sections thus found are not less than those required by the dead and live loads alone. In Chicago, where 18,000 psi is the basic unit stress for tension, an excess of 33 1/3 per cent is allowed for combined stress, thus permitting in both New York and Chicago a working stress of 24,000 psi tension for combined loads. This same unit stress is followed in the code recommended by the National Board of Fire Underwriters. The Recommended Building Code Requirements for Working Stresses in Building Materials, 1926, of the U.S. Bureau of Standards, favor an increase of 25 per cent based on 18,000 psi in tension."

After listing the requirements of several codes, Fleming recommends the use of 24,000 psi tension on the net section for "stresses due to wind loads combined with live and dead loads, or for members taking wind stress alone."

In 1940, an ASCE Subcommittee produced a report entitled *Wind Bracing in Steel Buildings.*[9] In it is found the first departure from the all-inclusiveness of the 1/3 stress increase and possible support for answer no. 3 above.

> "For members or details subject to wind stress only, except rivets and bolts, the permissible stress should be the same as that allowed for dead load or for dead and live load. For members subjected to stresses arising from the combined similar action of wind and other loads, and for rivets and bolts subject to wind stress, the wind stress up to 33 1/3 of the other stresses may be neglected, the excess wind stress being considered as equivalent to an added live load stress, provision being made for it at the basic working stress for dead load and live load only."

Note that rivets and bolts are permitted a stress increase for wind only.

In 1947, the first edition of the American Iron and Steel Institute *Light Gage Cold-Formed Steel Design Manual*[10] was published. The pertinent provisions followed the AISC specifications in permitting a 1/3 increase for combined stresses and for "wind or earthquake only."

A quartet of British authors[11] state that a 25% increase is allowed and then go on to make a good argument for answer no. 1 above:

> "By Clause 25, the normal permissible stresses in the members may be increased by 25 per cent in cases where such increases are due solely to the stresses induced by wind. This higher working stress is allowed *because of the transient nature of the load* and also because the *structure is of sufficiently elastic nature to allow it to absorb such transient loads without permanent defects.*"

In a more recent publication, this insight is offered by McDonald:[12]

"It should be noted that a very *high wind load is a comparatively rare occurence* and that the *design wind speeds specified in most codes of practice may never actually occur* in the life of a structure. For this reason most structural codes allow a 25% (sometimes as much as 33%) increase in permissible stress for wind loadings."

More evidence for answer no. 1 is found in *Building Construction Handbook,*[13] Chapter 6 by Stetina:

"For wind or earthquake forces, acting alone or in combination with the design live and dead loads, allowable stresses may be increased one-third. The increase is allowed *because wind and seismic forces are of short duration.*"

This position is also supported by McCormac:[14]

"The maximum wind and earthquake pressures for which design is made occur at large intervals of time and then last for only relatively short periods of time. It, therefore, seems reasonable to use higher allowable stresses, such as the one-third AISC increase, for lateral forces than for the relatively long-term gravity live loads."

A dissenting opinion is offered by White, Gergely and Sexsmith:[15]

"In recognition of the highly unlikely occurrence of maximum wind or earthquake loads simultaneously with the full value of other live loads, codes generally allow a 33% increase in allowable stresses under these load combinations."

CONCLUSIONS

So what may be concluded from the foregoing evidence? The preponderance of literature on the subject supports answer no. 1. That is, the ⅓ stress increase is allowed because of the "transient nature" of wind; because wind loads "are intermittent and seldom reach their maximum"; because a very high wind load is "a comparatively rare occurrence" and "may never actually occur"; because wind forces are of "short duration."

I found no support whatever for answer no. 2 and only two references (9 and 15), which support answer no. 3.

It may be argued that this practice has its root deep in our engineering tradition, in a time when wind loads were not so well understood; that modern methods of applying wind loads account for some of the factors which were used to rationalize the stress increase, such as "short duration" and "rare occurrence".

Modern wind codes are based on better meteorological information and wind tunnels which accurately model the boundary layer. A "rare occurrence" in wind velocity is now programmed into the design pressure selection process in the form of a Mean Recurrance Interval Map. The "short duration" aspect is now accounted for by gust factors. So maybe the ⅓ stress increase is no longer appropriate.

Let's think about that a minute. The gust factor as defined in ANSI A58.1 represents anywhere from a 30% to a 120% increase in the basic pressure, depending on the exposure type. If wind loads are increased by 30%, and then the ⅓ stress increase disallowed, the net effect is a load that is increased by 70%. Modern wind codes may have higher loads than we used to use on some parts of a structure, such as corners, eaves, and ridges, but may have lower loads on other parts. The resultant *should* be about the same total load as we have always used, but distributed differently and more properly suited to location. On the basis of the foregoing, I can see no valid reason why modern wind standards with gust factors and mean recurrence intervals and peak coefficients should not continue to permit the designer to use the ⅓ stress increase.

However, the intent of this search was not to try to justify the present use of the traditional ⅓ stress increase, but to determine its origin and just why designers a few decades ago *thought* it was justified. I will leave it for others to decide if the conditions which made the practice appropriate years ago are still valid today.

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