PDHonline Course S271 (3 PDH)

Simplified Principles of Wind Analysis

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Simplified Principles of Wind Analysis

Ruben A. Gomez, P.E.

1.0 Introduction

For a regular structural engineer to master the concept of Wind Analysis, it takes countless hours of classroom time, study of innumerous volumes and laboratory test experience to have a clear understanding of this matter so it can be applied to practical cases found in the course of his daily practice. The principles of Vibrational Theory, Structural Dynamics, Material Behavior as well as the Nature of Wind itself must be clearly understood as a prerequisite to use the common wisdom of the currently preset empirical rules of design.

Even if the analysis is performed by using commercial computer programs so abundant nowadays, the design engineer must possess the basic knowledge and experience to decide which is the right way to proceed, furthermore and above all, he must have the ultimate tool assisting any skillful engineer, *the notion of the result* (see Appendix A).

As excerpts and highlights of a lecture presented by the author to a scientific audience in April 1999, this paper makes an attempt to bring those intricate principles together in a simplified manner and by intensively using graphic aids to improve memory retrieval, while avoiding the complexities of formula derivation which is the main deterrent to a large number of engineering practitioners. Nevertheless, the content has been updated to reflect evolutionary changes and current state of the art.

2.0 The Nature of Weather

The energy of the Sun falls over the surface of the Earth in the form of sunlight in varying angles thus heating the regions of the planet very differently. Those differences in temperature lead to different types of weather conditions.

In addition to temperature differentials, there are some other factors which also affect the weather, such as, atmospheric pressure, ocean currents, rainfall, cloud formations and wind. In general terms, low atmospheric pressure usually brings with it rain and stormy weather, while high atmospheric pressure leads to calm and sunny weather.

3.0 The Nature of Wind

The unequal heating of the Earth's surface also causes the wind. When the air over a given region becomes warmer and therefore lighter, it rises and as result of the rising, cooler air moves in from another area to take its place. This air movement will be proportional to those temperature differentials and is what we perceive as *wind.*

Atmospheric pressure, as related to temperature, will also and directly affect the movement of wind, which will flow from a region of high pressure to one of low pressure. The pressure differential between the two regions will determine the velocity of wind flow. A small difference will create a breeze while a large difference may lead to a storm.

Wind velocity is affected by the friction contributed by the irregularities and topography of the Earth's surface, such friction is largest at ground level and diminishes as the elevation above grade increases and becomes negligible for heights of about 1,500 ft. In an urban exposure where at a given time wind could be measured say at 30 MPH, on a point 1,000 ft directly above, that same wind could sustain a velocity of over 100 MPH. That particular zone of the atmosphere from ground level to an elevation of 1,000 ft. is the one of particular interest for the design engineer since most buildings are within that range of height.

Although in most graphic representations wind is shown as a planar phenomenon with a horizontal motion and depicted by its normal and tangential components, air motion is in reality a three-dimensional event when we consider its vertical motion, which is paramount for the cloud formation and therefore rainfall occurrence. All those forces have both measurable magnitude and direction, therefore being susceptible to representation as vectors.

If we represent over the map of a given region all points of equal barometric pressure and connect them together, the resulting contours would be the so called *Isobars.* The atmospheric (or barometric) pressure is expressed in *milibars.*

At heights of over 1,000 ft. where the friction of the Earth's surface has little influence, air flow is seen as affected by three main factors: a) atmospheric pressure, b) the centrifugal force generated by the isobars, and c) the *Coriolis Force* as a result of the rotational movement of the planet. The magnitude of the latter force is a variable going from a zero value at equatorial locations to a maximum value at the North and South Poles.

All those above described vectors are shown on the enclosed Figure 3.1 depicting the forces acting on an unitary volume of air trapped in between isobars. On that figure F_p is the *pressure gradient,* F_o is the centrifugal force, while F_c is the Coriolis force. Those three forces are related by the formula:

$$\mathbf{F_p = F_o + F_c}$$

As can be deducted the *pressure gradient* is balanced out and therefore equal to the sum of the *centrifugal* and *Coriolis* forces.

The fourth vector not mentioned above, but most meaningful to the engineer's work is the actual *wind force* which happens to be tangential to the isobars, as shown on said figure.

As indicated above, prevailing conditions on the surface of the Earth, such as topography, vegetation and buildings contribute to increase friction and therefore reduce wind velocity and its resulting pressure on new construction. Figure 3.2 shows a graph depicting how those terrain conditions will affect *design wind pressure* within a zone between zero to 1,000 ft. above the ground surface.

When air flows over and around a given building, the wind pressure generates both external and internal pressure on walls and roof. Furthermore, the presence of said building by itself within the air current will tend to change original wind velocity and direction, and last but not least, will create a certain amount of *turbulence* depending on building shape and corner form and sharpness. Figure 3.3 depicts some common building forms and how they affect the pattern of air flow and the resulting internal pressure conditions.

Considering what we have seen so far, we must conclude that there should be at least three factors of interest to the design engineer, such as:

1. The most important of them, *pure wind velocity* directly depending on physical characteristics: geographic location, height above ground, topography and degree of exposure.

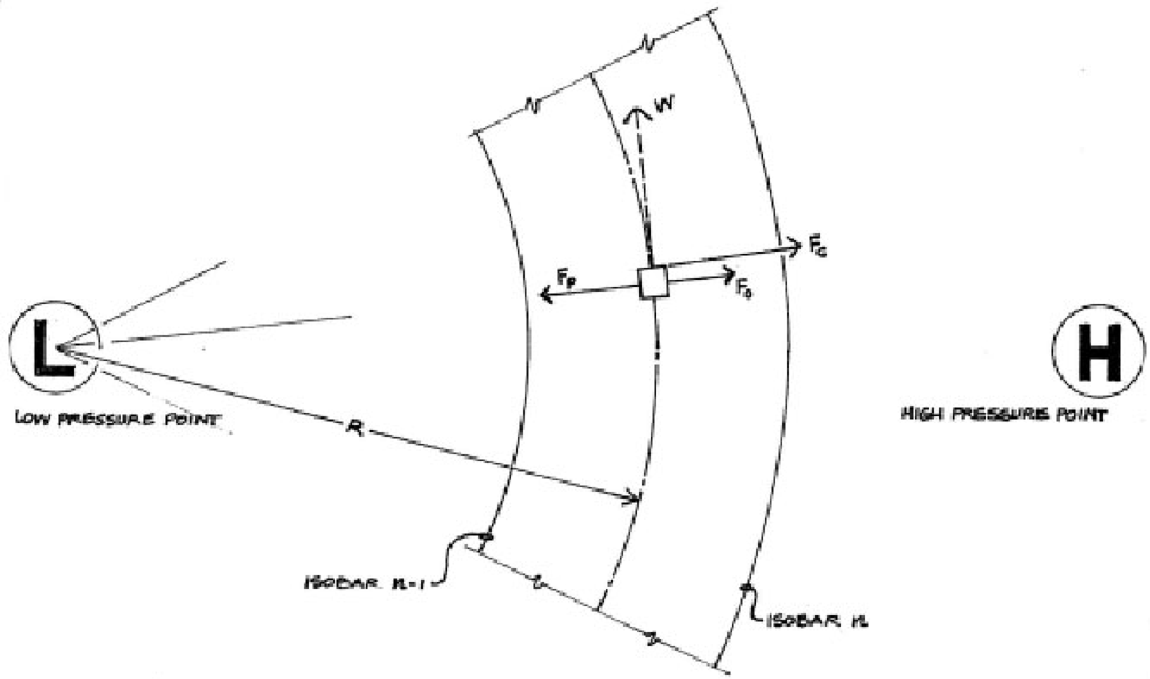


FIGURE 3.1 FORCES ACTING ON AN UNITARY AIR VOLUME BETWEEN TWO ISOBARS.

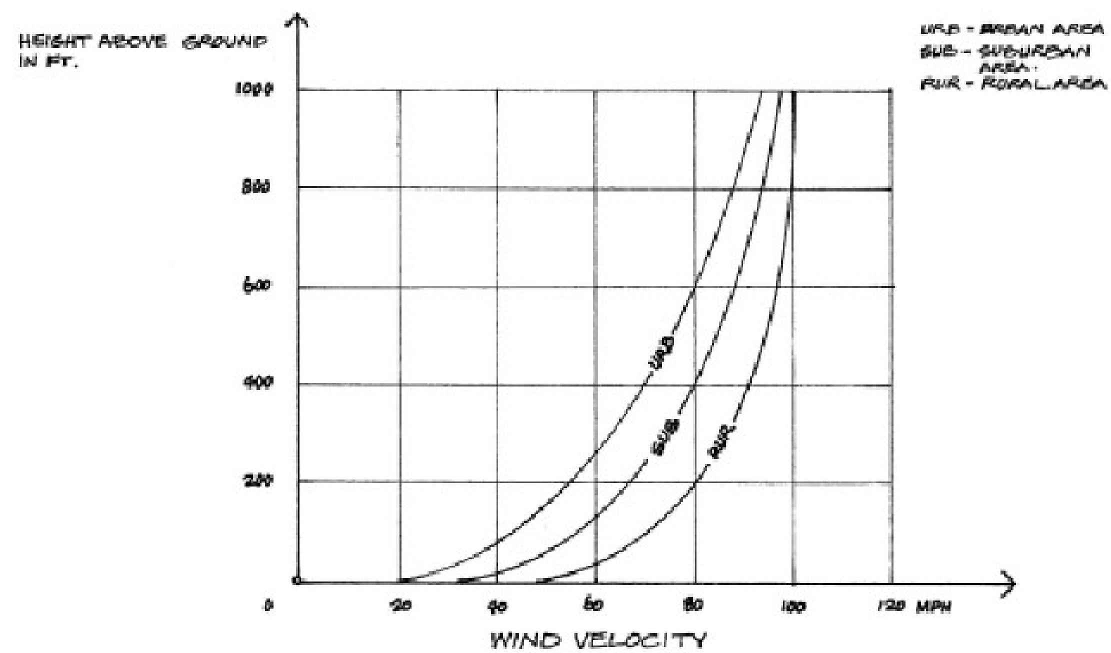


FIGURE 3.2 HOW TERRAIN CONDITIONS AFFECT WIND PRESSURE

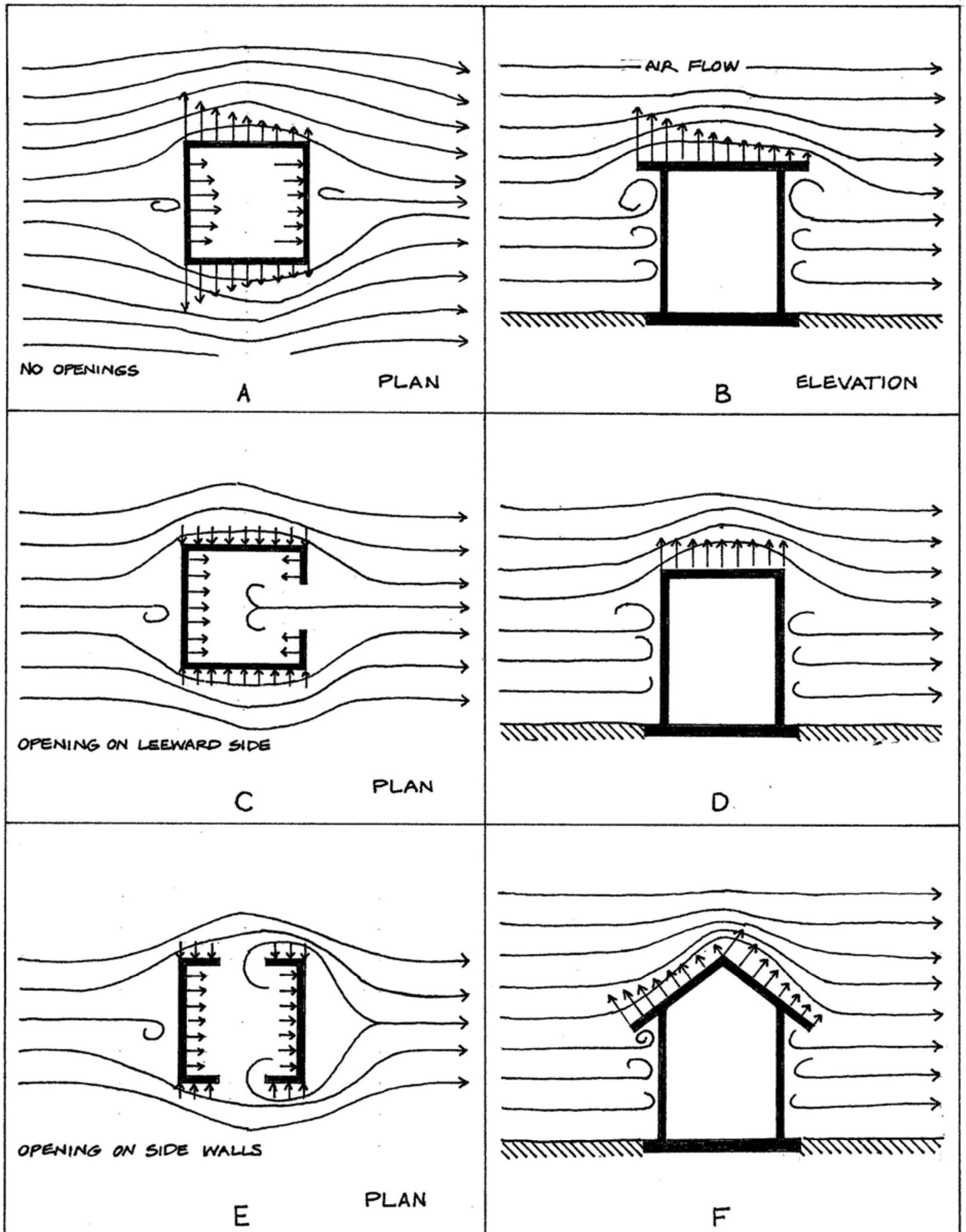


FIGURE 3.3 HOW SHAPE, ORIENTATION & OPENINGS AFFECT WIND STRESSES.

2. Changes in wind velocity as determined by the form, size and proportions of the structure under consideration.

3. The potential of generated *turbulence* is also dependent on shape, shielding factors, orientation and corner conditions of the proposed structure.

Figure 3.3 gives us a mix of conditions showing how shape, form, roof overhangs and whether that roof is either flat or sloped, and last, the presence of openings as they not only alter the flow of wind but also the pressure distribution around and within the structure.

Condition "A" depicts an enclosed and sealed structure, quite unlikely in normal practice but yet important to gives us that point of reference. By the way, not with the intention of confusing the main issue here, but for the purpose of pure wind analysis and design, it must be emphasized here that structures are ideally conceived as enclosed structures. Unfortunately, openings are needed for access, natural light and ventilation, and there is one of the big problems of modern construction. Under high wind conditions failure always start by the fenestrations (windows and doors) followed by failure of the roof components.

When window and doors fail to adequately resist wind pressure, the building becomes an "open structure", much as shown on Conditions "C" and "E" on the same Figure 3.3, and pressure distribution and therefore stresses on walls and roofs become totally different and if such stresses have not been taken into consideration a structural collapse becomes imminent.

Upwards pressure (or wind uplift) is indeed a fascinating event which should be well understood by any engineer who adds wind analysis to his practice. It all starts with Bernoulli's Theorem about *fluid flow* and his *law of conservation of energy* predicating that when a moving solid is immersed in a fluid (air in our case), there are two conditions which shall be met:

#1- In any given moment and any given point, the resultant of fluid pressure must be equal to the resultant of *kinetic energy.*

#2- If the fluid pressure changes due to a change in velocity, the resultant of kinetic energy must also change in the same amount but with reversed polarity.

The validity of Bernoulli's principles comes demonstrated by the airplane's wings and therefore why such an aircraft is able to fly for as long as there is a sustained velocity.

Figure 3.4 on picture "A" is depicted an airplane wing which is designed and built in such a way that the upper face is longer than the lower face. When the plane is in motion its wings will displace the air filaments in such a way that some of them will flow over and the rest under, as shown on picture "B". Air flowing over the wing will have to travel longer, however, in order to meet Bernoulli's rule and maintain pressure balanced those filaments not only have to meet at the tail of the wing, but those going over the wing will have to have a higher velocity to be able to meet those going under the wing. That velocity differential in order to be balanced will have to produce a set of uplift forces forcing the wing, and therefore the plane, in an upwards motion (see picture "C" on the same figure).

Another two applications showing the validity of Bernoulli's Theorem are depicted on Figure 3.5 where picture "A" consists of a bus shelter with a sloped roof which must be secured by proper strapping to the supporting structure so as to prevent being blown away by uplift forces generated by hurricane winds.

Picture "B" on the other hand is a scene we all have seen, it shows a convertible automobile being driven at a certain speed enough so the acceleration of the air filaments flowing over

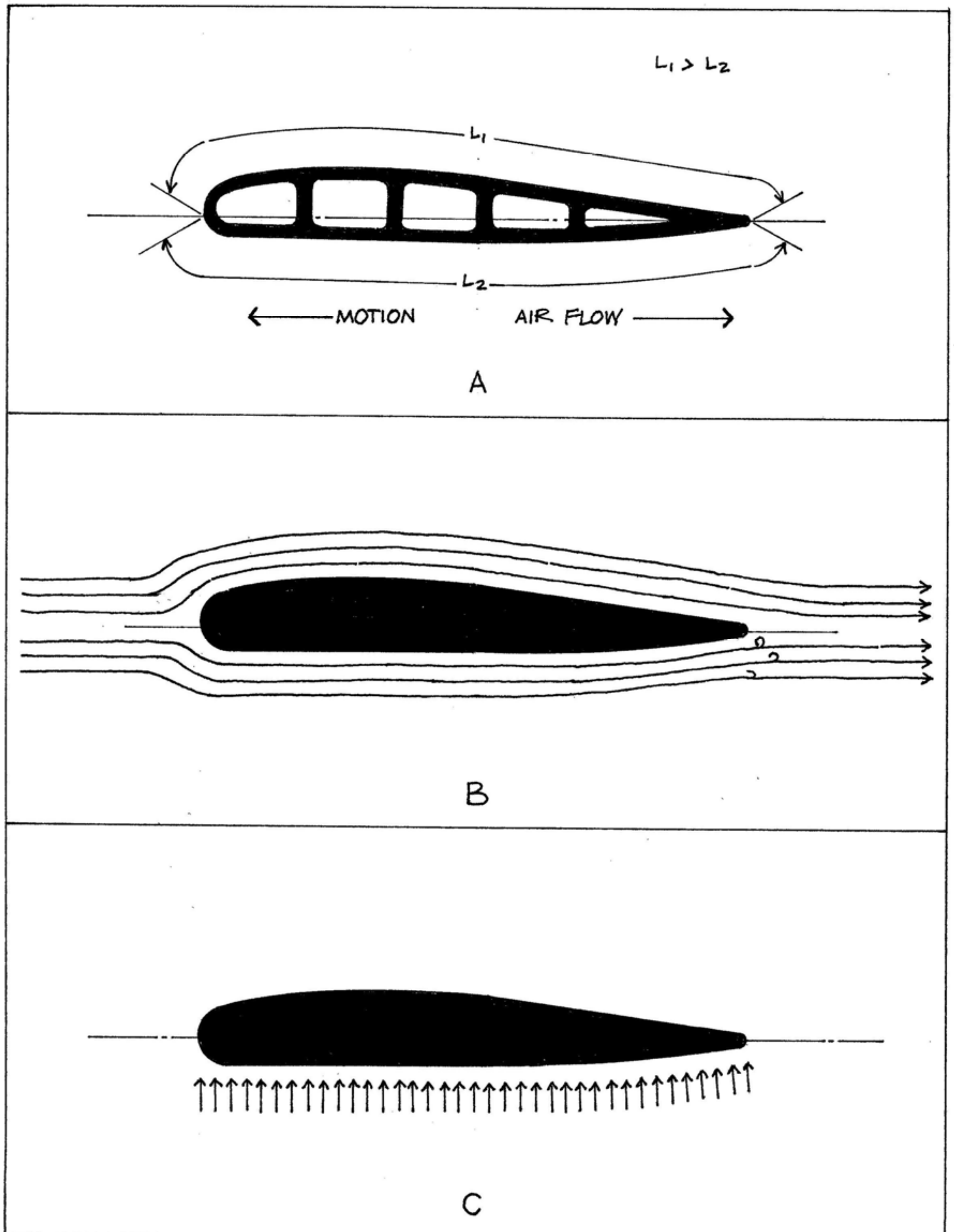


FIGURE 3.4 TO SHOW WHY AIRPLANES FLY.

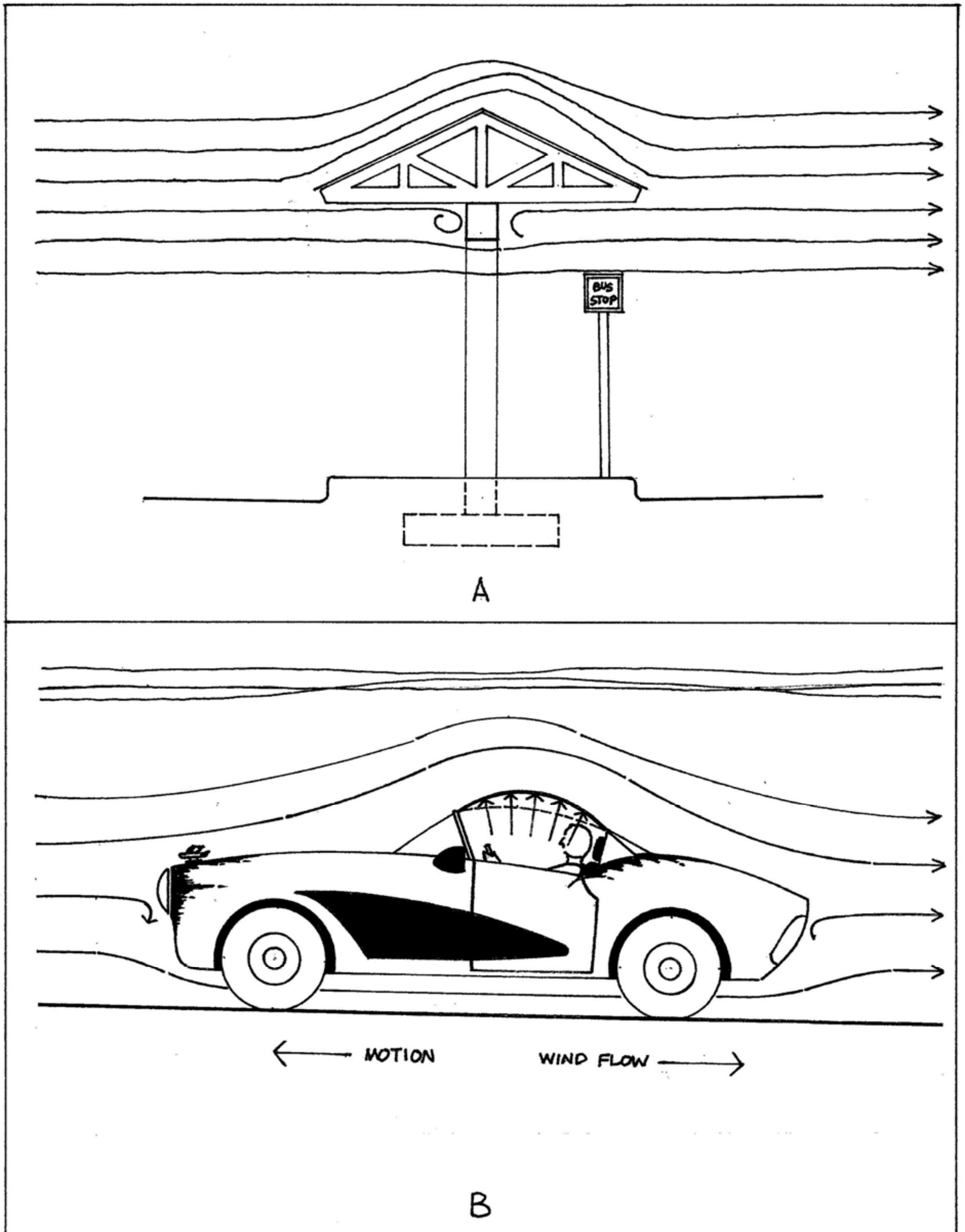


FIGURE 3.5 MORE APPLICATIONS OF BERNOULLI'S THEOREM.

its canvas roof would create an uplift force tending to swell up the same out of its intended shape.

Meteorology in general and particularly the art of hurricane prediction has come a long way during the last twenty years. A hurricane can be predicted several weeks ahead of time and within a week's time the path can be determined with a large degree of precision.

The time span we are referring to is not enough for the design engineer to make adjustments, however, it is enough for reasonable preparations to be made to protect human life and minimize damage to existing structures. Once the hurricane path is predicted, we can even know which parts of a given building or structure is going to be battered by the wind and its accompanying flying debris. Also, as an after-the-fact valuable information, the forensic engineer can make a determination of what the conditions were during the passing of the hurricane winds.

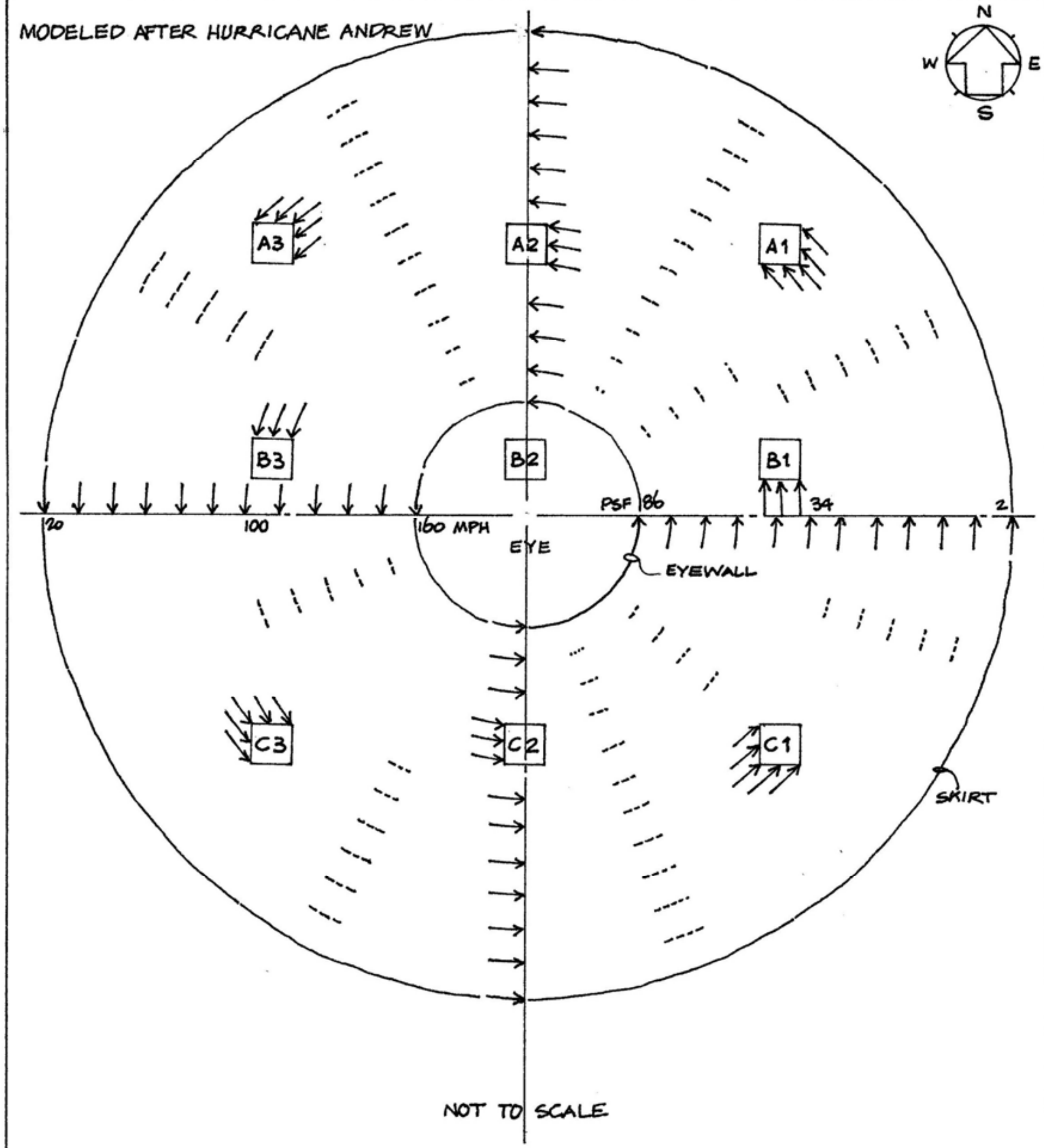
Granted, there are all kinds of hurricanes with a varied range of [rotational] wind velocity anywhere between 74 to 170 MPH, overall spread [diameter] from 80 to 400 miles and eye diameter from 20 to 100 miles, as well as the storm's translational speed and barometric pressure. However, all those characteristics are well known a week ahead of time within the prediction period.

In the early 1990's, immediately after Hurricane Andrew, we in our office, developed a simple diagram that proved to be quite helpful in damage analysis, damage classification, damage prediction and other areas of forensic engineering of interest for the *insurers* (mostly insurance companies).

Such diagram is reproduced here in as part of Figure 3.6 and is most useful in predicting wind velocity, wind pressure according to position and orientation of the building in question. The diagram shows an idealized hurricane perfectly symmetrical about its NS and EW axis, which admittedly is rarely the case in common occurrence when they are, more often than not, heavily eccentric towards the NE and NW quadrants, consequently, the forensic engineer's good judgment and experience should prevail during its application by adjusting the diagram in accordance to the actual conditions.

The idea is to predict wind pressure, wind angle of attack and vulnerability of a given building by working three possibilities of event depending on whether the subject structure is located North of the eye as shown on (A1- A3), South (C1-C3) or right passing through the eye of the storm (B1-B3). Although in practice the hurricane is in motion and the buildings are static, for the practical purpose of using the diagram the assumption is quite the opposite, the hurricane remains stationary and the building moves along the anticipated hurricane path. Hurricane winds are represented to rotate in a counterclockwise motion as the typical condition on the northern hemisphere. Lastly, it should be noticed that the diagram was prepared to simulate conditions similar to those observed during Hurricane Andrew on its path over the southern tip of Florida in August of 1992, with a wind velocity (right at the eye's wall) of approximately 160 MPH which calculated according to the current South Florida Building Code at the time, gave us a design pressure of about 86 PSF for a prismatic building.

A brief scrutiny of such diagram would bring over the following observations, when the structure is located on the North side of the eye, it would not sustain pressure from westerly winds, when on the South side of the eye, no wind should be expected coming from the easterly side, and lastly, when the eye would pass right over the subject structure, no appreciable wind was to be expected from either the East or West sides. A small tool that would help the forensic engineer in figuring out a likely mechanism of failure if so needed.



NORTHERN HEMISPHERE
- BUILDING A WILL NOT BE SUBJECT TO WESTERLY WIND.
- BUILDING B WILL NOT EXPERIENCE EITHER WESTERLY OR EASTERLY WIND.
- BUILDING C WILL NOT BE SUBJECT TO EASTERLY WIND.

FIGURE 3.6 IDEALIZED DIAGRAM OF A HURRICANE.

4.0 ABBREVIATED NOTIONS OF THE VIBRATIONAL THEORY

The study of vibrations has a certain fascination for most of us because we live in a world filled with vibrations which constantly manifest themselves in our daily living in the form of movement, colors and sounds.

As an example of these experiences, let us consider for a moment the analogy of the pebble falling over the still waters of a pond on a serene weather day. When it so happens and such pebble impacts the water surface thus breaking the equilibrium in the molecules of the liquid, the imparted energy creates a *kinetic wave* that displaces in a concentric and centrifugal manner. As the inertial effect in the water produces a certain level of resistance to the movement, the wave is resisted by a *damping effect* which tends to increase the *period of vibration*, and at the same time, to reduce the *acceleration* imparted to the water molecules in motion. In a few more seconds, the wave ceases, all movement stops and the pond surface returns to its original stillness.

Figure 4.1 gives us a graphical account of the common phenomena we have just described above. In fact this would be a good subject of research to further investigate the traits of the kinetic wave by using a large plastic container with permanent graduations clearly marked on the front wall of the container, a timer and a multiple lens camera on a tripod and a high speed film, so as to record the different phases of the event. With such equipment it would be possible to use a variety of weights in half ounce intervals so as to determine their effect on the period of vibration (commonly designated as **T),** the wave amplitude **a**, the frequency **f** and the damping **C**. Once those variables are tabulated and the corresponding curves drawn accordingly, interesting conclusions could be arrived at. Furthermore, the research could be expanded to cover different media of propagation in addition to water, such as alcohol, oil and other liquids of different *viscosity* to enable widened available data between the values of **W** and **C**.

On this same figure above referred to, we can appreciate graphically where the *period of vibration* is the interval of time **T** which takes the wave to displace from peak to peak, while the diminishing amplitude **a** is measured from the positive (or negative) peak to the reference axis.

Another simple analogy to help visualize the principles of pure vibration is by using the *pendulum,* consisting of a weight attached to the free end of a flexible steel bar, while the other end is adequately fastened to the top of a laboratory table, such system is shown on Figure 4.2. If the table is suddenly shaken an inertial force will be induced into the weight of the pendulum and it will swing in an oscillatory mode from one end to the other. The distance measured from the still position **A** to any of the two extreme position of oscillation **B** or **C** is what is called *oscillatory amplitude.* The time interval required for the weight **W** to complete one complete oscillation, meaning from **B** to **C** and back to **B**, is called the *natural period of vibration.* In the same way, all vibratory bodies have their own natural period.

The vibration mode above described could well be the first tremor of an earthquake or the first gust of a hurricane and constitute a form of *free vibration.* If during the swings back and forth an exterior lateral load is induced into the weight of the pendulum, the resulting altered motion would be a *forced vibration* as it will be seen below once more.

As part of the event and immediately after the first shake or the first gust, similar events will take place which will induce additional loads to the system already in a state of excitation. Those additional loads would then induce a mode of forced vibration into the system already in motion.

When a *free body* vibrates without interference from external causes, it is said that it is vibrating in a *free mode* and according to its *natural period.* On the other hand, when such vibration is

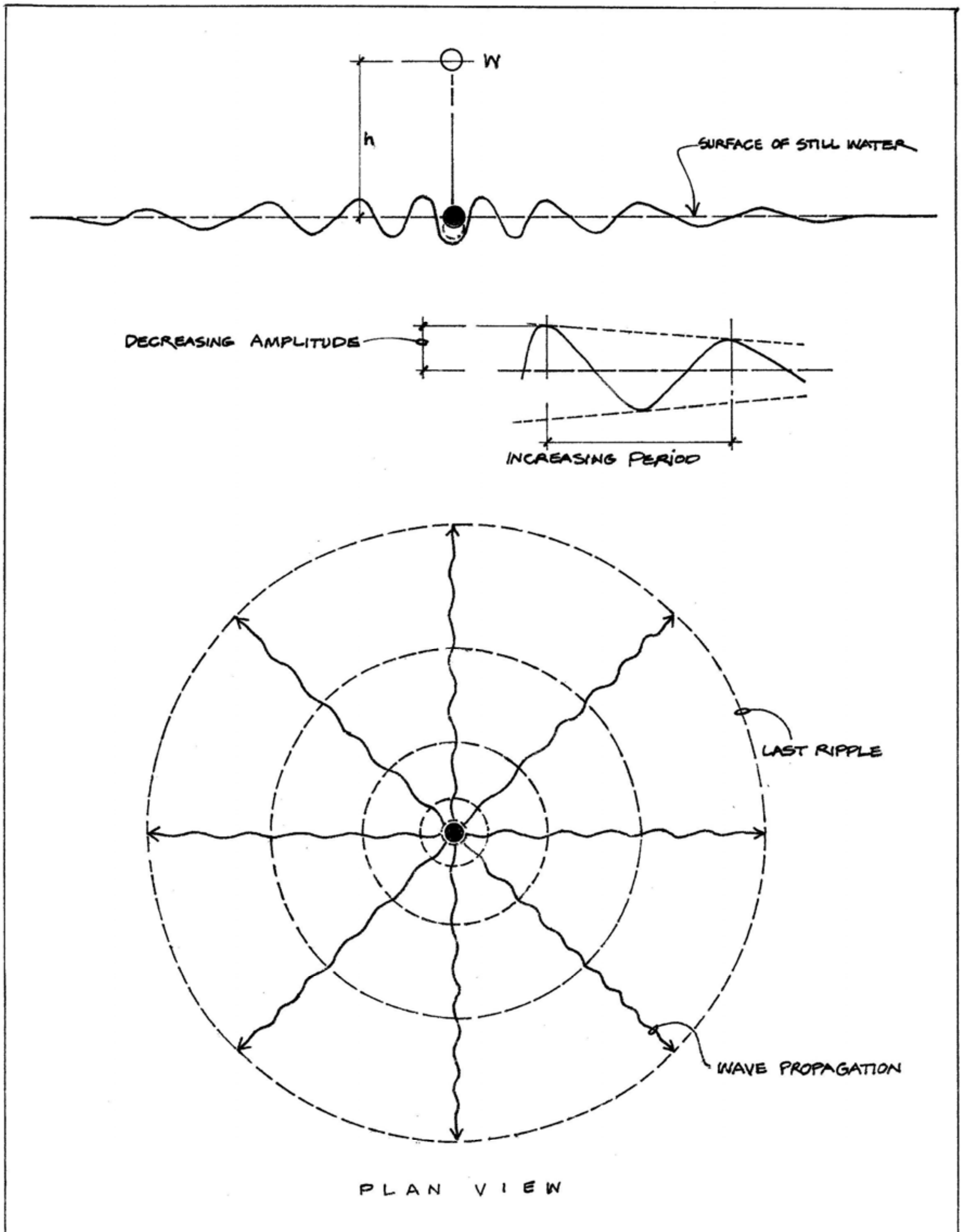


FIGURE 4.1 ANALOGY OF THE PEBBLE FALLING IN THE POND.

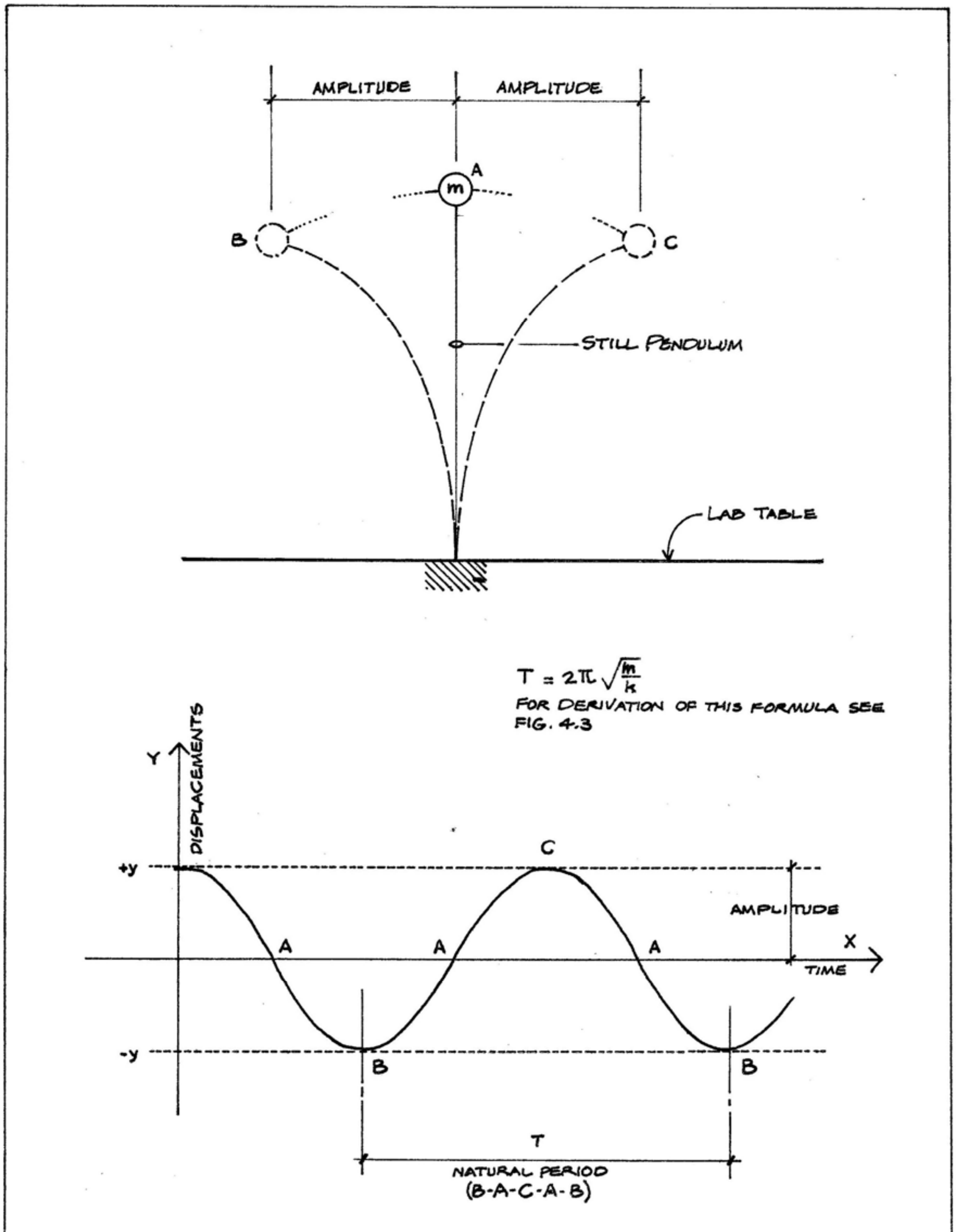


FIGURE 4.2 ANALOGY OF THE PENDULUM.

influenced by external forces tending to modify its free vibration, then it is called *forced vibration.* Further, when those external forces are such that would tend to induce and maintain a frequency equal to that of the *natural frequency* of the vibrating body, such mode is known as *resonance,* which is a very feared condition, because when maintained for a period of time it could cause total collapse and self-destruction. The derivation of the natural period and frequency formulae is shown on Figure 4.3.

The following concepts on vibrational theory will be limited to the oscillatory motions of the type known as *harmonic simple,* which mostly are those commonly used in solving the problems found in structural dynamics and building design.

The study of the forced vibration phenomena in multistory buildings is quite tedious and complex. As a matter of simplification, it must be added here that the three mainly significant modes of vibration have been identified and are frequently referred to as the *Eigen Vectors* in honor of its conceiver. Although they do not occur simultaneously, they do take place in a rapid sequence to the point that some of the generated stresses may become additive depending on the location within the height of the building structure. The shape of those modes of vibration reminds the contortions of a whip. They are shown herein on Figure 4.4 for illustration purposes.

DERIVATION FOR THE NATURAL PERIOD AND FREQUENCY

Nomenclature:

f = frequency (cycles per second)
g = acceleration of gravity (32 ft. per second squared)
k = spring coefficient
M, m = mass = W/g
T, t = period (seconds)
W = weight (lbs)

Since a complete cycle takes place on every angular increment, the angular frequency is:
$\omega = s/t = 360/t = 2\pi/t$

Because: $\pi = 180$ (degrees)

thus: $\omega t = 2\pi$

therefore: $t = 2\pi/\omega$ (seconds).

On the other hand, the angular frequency is:
$\omega = \sqrt{k/m}$ (radians per second)

Therefore, the natural period is equal to:
$T = 2\pi\sqrt{m/k}$ (seconds)

As we already knew, frequency is the reciprocal of the natural period, thus:

$f = 1/T$ (cycles per second)

FIGURE 4.3 DERIVATIONS OF NATURAL PERIOD AND FREQUENCY FORMULAS.

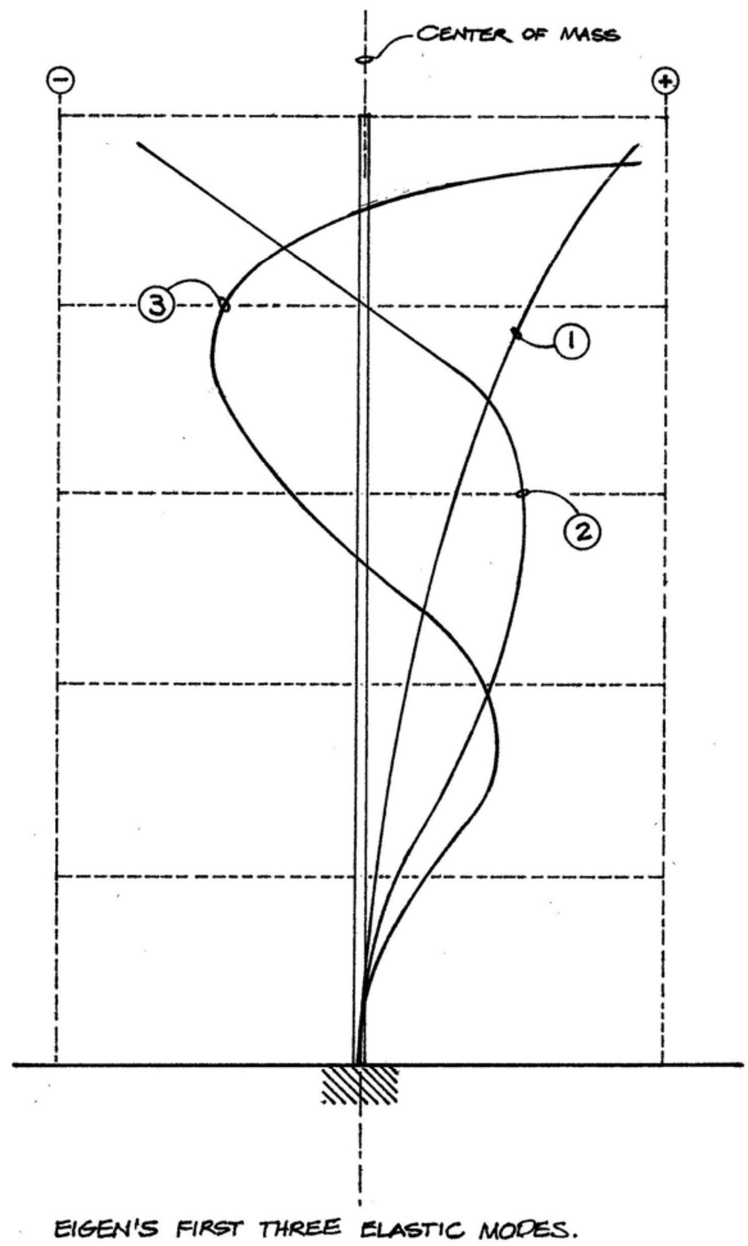


FIGURE 4.4 EIGEN'S VECTORS

5.0 DESIGN CONSIDERATIONS

For over a hundred years there has been plenty of debate between the followers of the two main schools of thought *rigidity vs. flexibility.* Although flexible structures behave better under sustained high winds, the perception amongst the occupants may be one of uneasiness due to the high levels of deflection especially on the high floors. On the other hand, a rigid structure by its own nature induces to higher stresses which at the same time begets increased rigidity. This is a challenge to the ingenuity of the design engineer in his search for a happy medium.

The concepts of flexibility and rigidity are defined in two different ways, firstly by the total deflection caused by the lateral loads. The product of dividing the total deflection by the building height determines the dividing line between the two. A value of 0.05 is such a boundary as shown on Figure 5.1. The second manner is based on the *natural period of vibration* which defines *rigid* structures as those with a natural period (T) between 0.10 and 1.20 seconds, larger periods define those structures designated as *flexible.* Please read more about this matter on the chapter's closing statement below.

Wind induced lateral loads on any given building structure can be resisted by using any of the following four (4) design systems:

a) **Rigid Frames,** currently called WSMF (welded steel-moment frame), where lateral loads are resisted by a welded multi-jointed frame of columns and beams. With this type of arrangement the resulting structure should be capable to carry not only the vertical loads, but also should be able to resist all lateral loads generated by either high winds or seismic motions.

In order to achieve such goal, the end of beams must be rigidly connected to the columns by welded connections and be able to handle all vertical and horizontal loadings without the assistance of braces or shear walls. The concept of continuity between columns and beams was achieved by means of welded joints which were intended to have the same strength, ductility and all other structural characteristics of its joining members.

In addition to its clean design and straight forward way to facilitate erection, there also was a great advantage in the freedom of interior space it allowed to the architectural designer, and in many instances even a larger freedom in the exterior design and appearance of the building.

Thousands of these buildings were erected during a period of 30 years after its first application in the early 1960's, then after the Northridge earthquake in 1994 problems started to get noticeable and that triggered a deeper examination which revealed compromising damages to hundreds of buildings, sustained damages that had remained hidden for some time behind the interior finishes.

Most of the sustained damage discovered in WSMF buildings was related to crushed and cracked joint welds, which compromised the safety and dependability of such joints to carrying the intended loads.

After that event, there have been improvements both in the design and fabrication practices as recommended by a joined effort between The Federal Emergency Management Agency (FEMA) and a group of entities such as the Structural Engineers Association of California (SEAOC), the Applied Technology Council (ATC) and others. Together they produced a comprehensive report in November 2000 titled "A Policy Guide to Steel Moment-frame Construction" which has led the way to improve design and construction of WSMF buildings.

Lessons were learned which now have been resolved and some of those solutions implemented have been added in terms of new requirements incorporated into the current building codes.

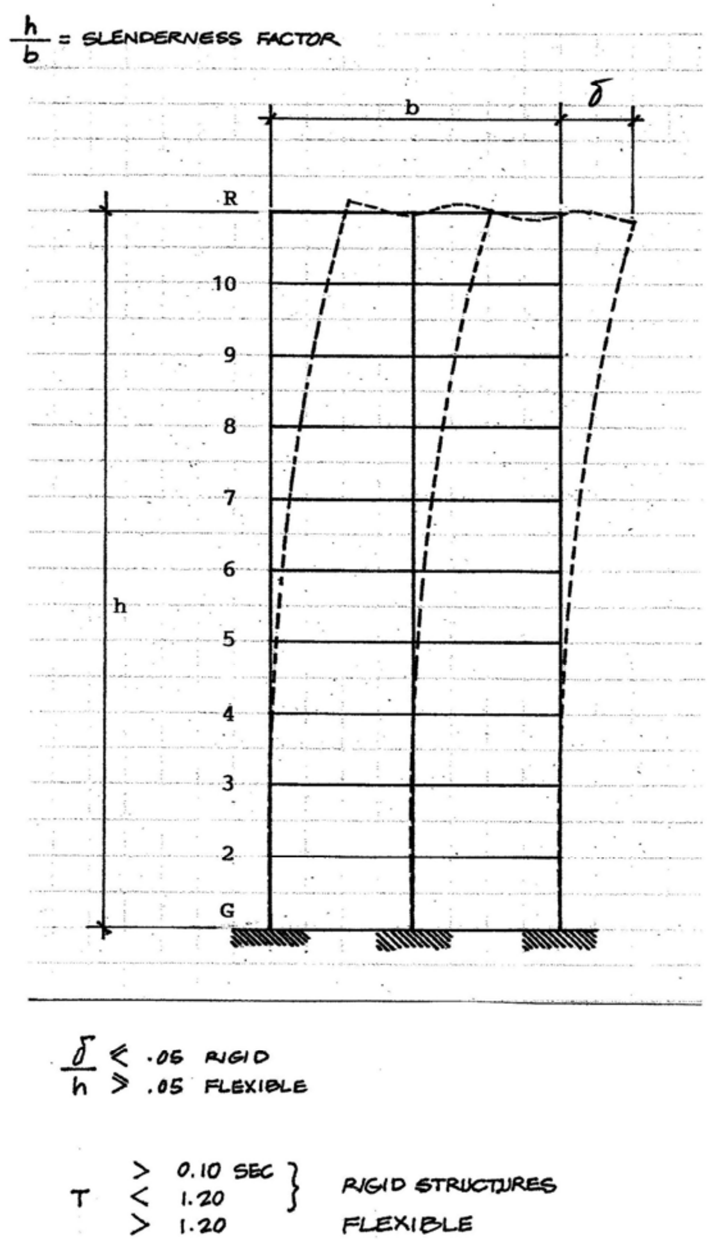


FIGURE 5.1 BOUNDARIES BETWEEN RIGIDITY AND FLEXIBILITY.

In spite of all those gains, it is important for the design engineer to carefully and cautiously detail his joints and accompanying specifications so to avoid a repetition of the mistakes of the past.

b) **Shear Walls**, where either all or a large portion of the lateral loads are solved by using reinforced concrete walls adequately located within the structural plan, acting as tall cantilevers held at the ground level. This solution remains popular and fairly safe for its application to wind design. A typical design case has been selected and added to this paper in Appendix B as a classic procedure of hand calculations for a five storied building with shear walls as a wind resisting solution.

c) **A hybridized solution combination of a) and b).**

d) **X-braced Frames**, similar to solution a), except that the welded moment connections are replaced by an "X" bracing system connecting the columns and beams at strategic locations (where the shear walls would be on solution b) and at every floor level.

The four above described solutions have been graphically represented on Figure 5.2. It should be noticed that elastic deformations have been somewhat exaggerated for the purposes of clarity. It is also important to realize that in the case solution marked as c), where the frame and shear walls are combined and interacting with each other, that the same interaction will produce secondary stresses of interconnection that must be taken into account and properly addressed.

Lastly, it is the responsibility of the design engineer to select to the best of his judgment, and depending on the type of building, material, slenderness and budget, which structural system would be best suited for the case at hand.

It is paramount that in the architectural conception of multistoried buildings, the principles of aerodynamics are given important consideration when deciding about the foot print and shape. On the enclosed Figure 5.3 examples of desirable and undesirable shapes have been defined for the consideration of the design architect.

Forms a) through c) are desirable because they take advantage of aerodynamics in such a way that improve air flow by reducing pressure and avoiding turbulence. For the opposite reasons, forms d) through f) tend to exacerbate the problems of friction and turbulence.

As it was indicated before, wind resistant buildings are designed as fully enclosed structures with a reasonable degree of air tightness and where the components of such enclosure, namely doors and windows are capable of resisting the predicted wind pressure, gusts and the impact of flying objects.

By the moment a building under wind pressure losses its door and windows (in spite of the structural engineer's best efforts to avoid it), it will be subject to conditions different from those it was designed for. If such scenario would take place the resulting structure would be subject to certain pressures, suctions, uplifts and turbulences difficult to predict, let alone the predicament to be placed on its occupants. It is for this reason that the design engineer has the unavoidable responsibility to properly design the anchoring of all door and windows in such a way they can resist all forces and further, be able to transmit them safely to the main structure.

When those doors and windows are made of glass, it is improbable that they could stand to the wind pressure and even less probable to resist the impact of flying debris. In most of those cases it would be more practical rather than to take the risk of fenestration failure, to choose one of the following two protective measures:

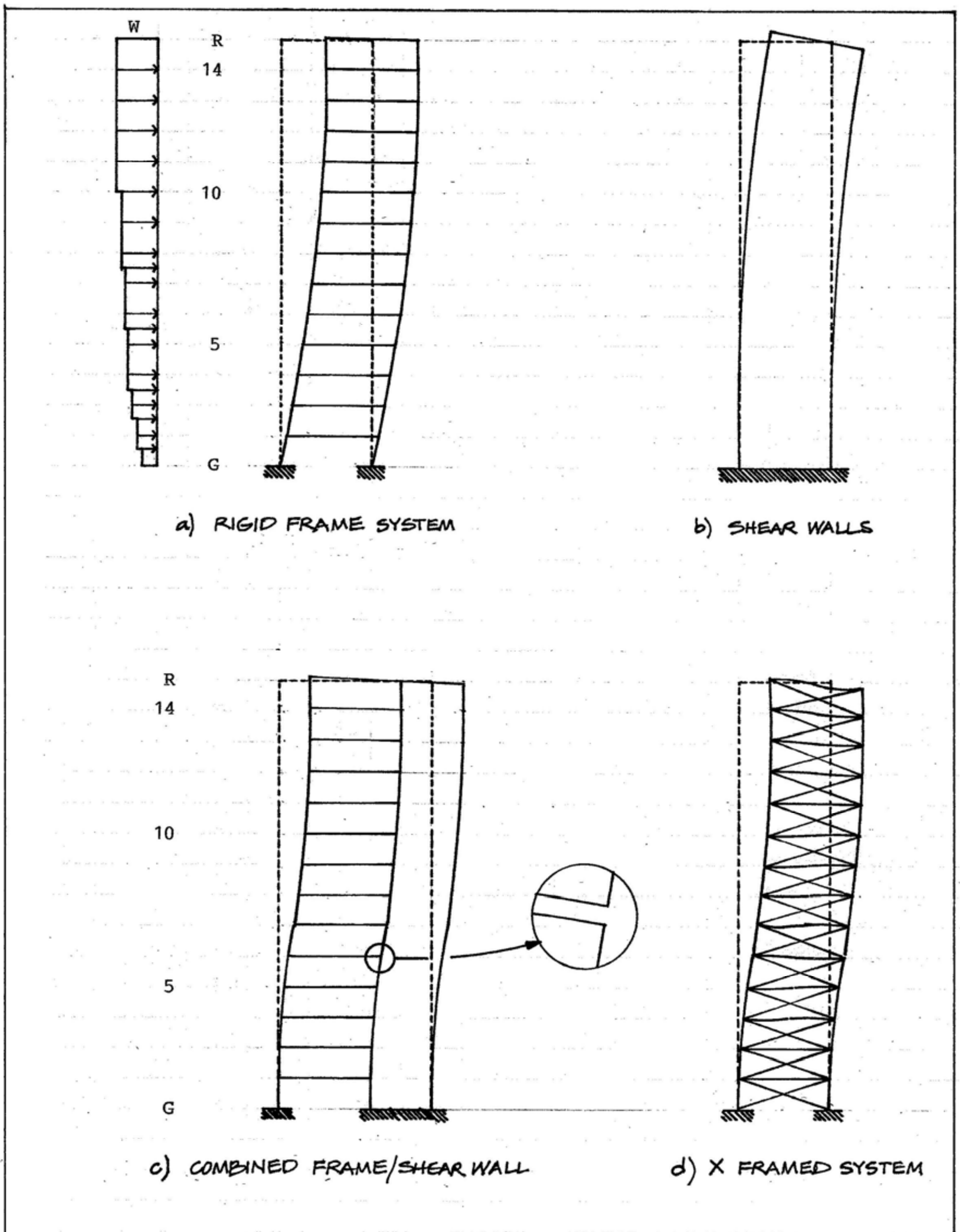


FIGURE 5.2 WIND RESISTING FRAMINGS

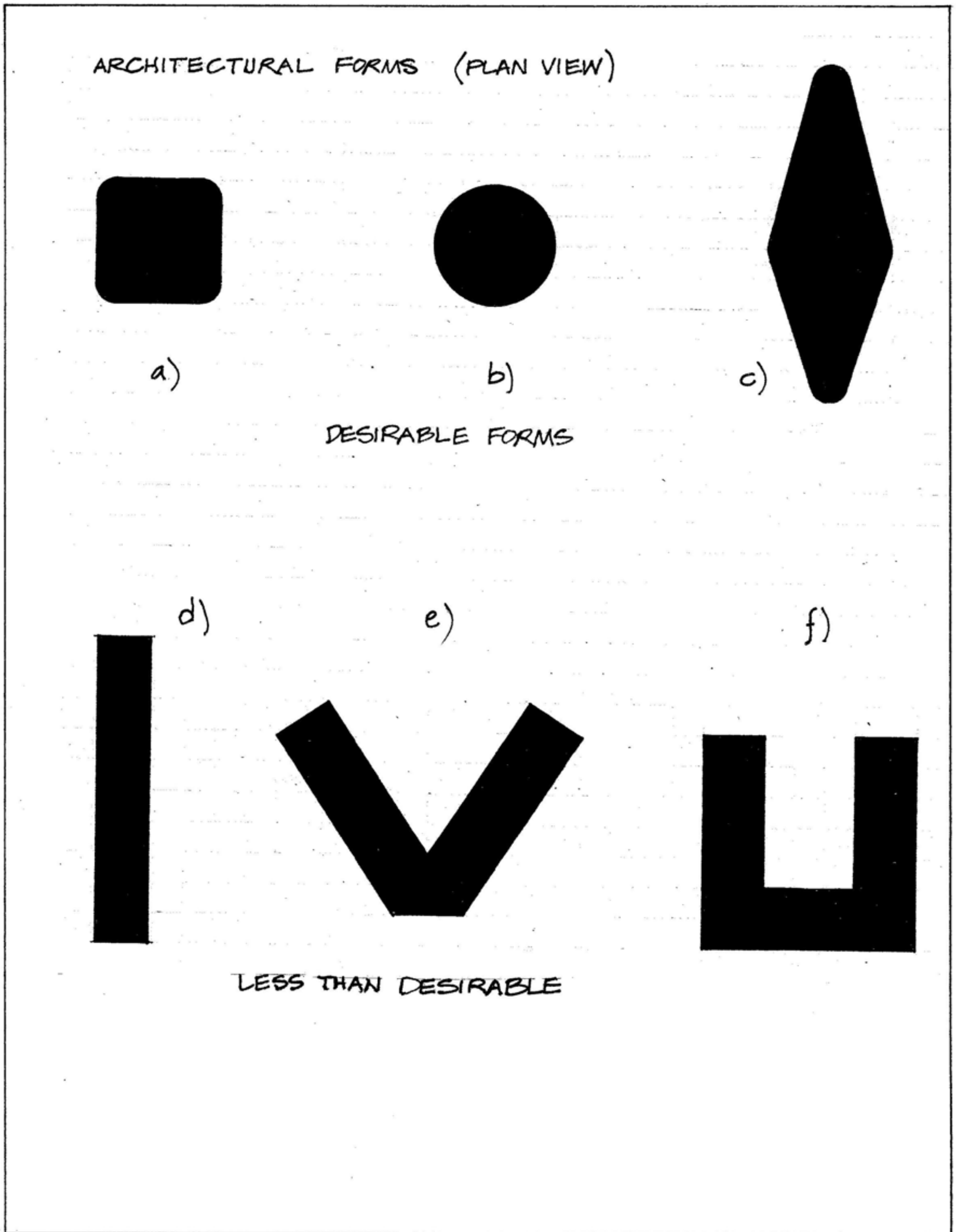


FIGURE 5.3 BUILDING FOOTPRINTS

#1- To provide on the exterior side of doors, windows, balconies and terraces as well, aluminum (steel, reinforced plastic or titanium) folding panels, either hand operated or motorized and remote controlled. Figure 5.4 shows a version of a current commercial folding shutter which is very effective not only in resisting winds of over 200 MPH, but also adequate to stand to the impact of flying debris. Most of its adequacy is derived from the geometry of its folded panels with their peaks a valleys, which make them extremely strong to horizontal forces as depicted on said figure.

#2- Either to place those openings in such a way that they are moved away from the direct path of the wind or have them protected by design integrated grilles such as those suggested on Figure 5.5. On picture a) of the same figure, it is suggested that windows be placed in a niche like wall projection having the front end protected by grilled concrete blocks, while b) suggests prefabricated grilled panels to be installed in front of window groups. On the other hand, solution c) shows doors and windows protected by means of creating a semi enclosed balcony or small porch in such a way that wind flow is deflected from the fenestration.

Going back to the concept of rigidity vs. flexibility, it must also be added that rather than to follow the concept of rigidity where the loads and their resulting stresses are entirely resisted by the “brute” strength of the material(s) employed, it would be much wiser to absorb the energy transmitted as result of the lateral loading’s action, by using the flexibility inherent in the structural system which could be further enhanced by allowing controlled displacements at the base of the framework.

With such an idea in mind and as a vision of the near future, Figure 5.6 depicts a solution borrowed from principles observed in mechanical mobile systems where rather than using the traditional and conventional concept of rigid connections to the foundation, instead, the framing is installed on rollers that would allow a certain degree of movement which is at the same time controlled by damping pistons or hydraulic shock absorbing devices conveniently and appropriately located to maximize their function and mitigate damage. Such and idea not only would dissipate large amounts of destructive energy but also could save a building from the disastrous consequences of extreme lateral loadings generated by either high winds or earthquakes.

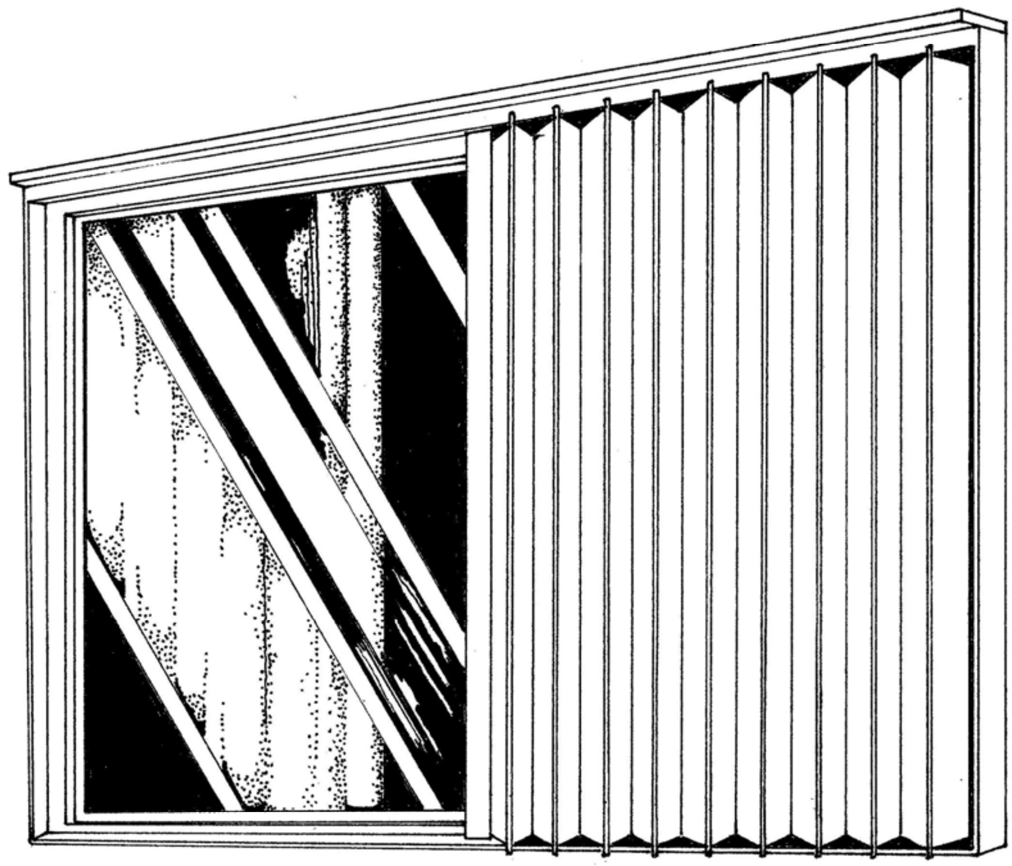


FIGURE 5.4 HURRICANE FOLDING SHUTTER

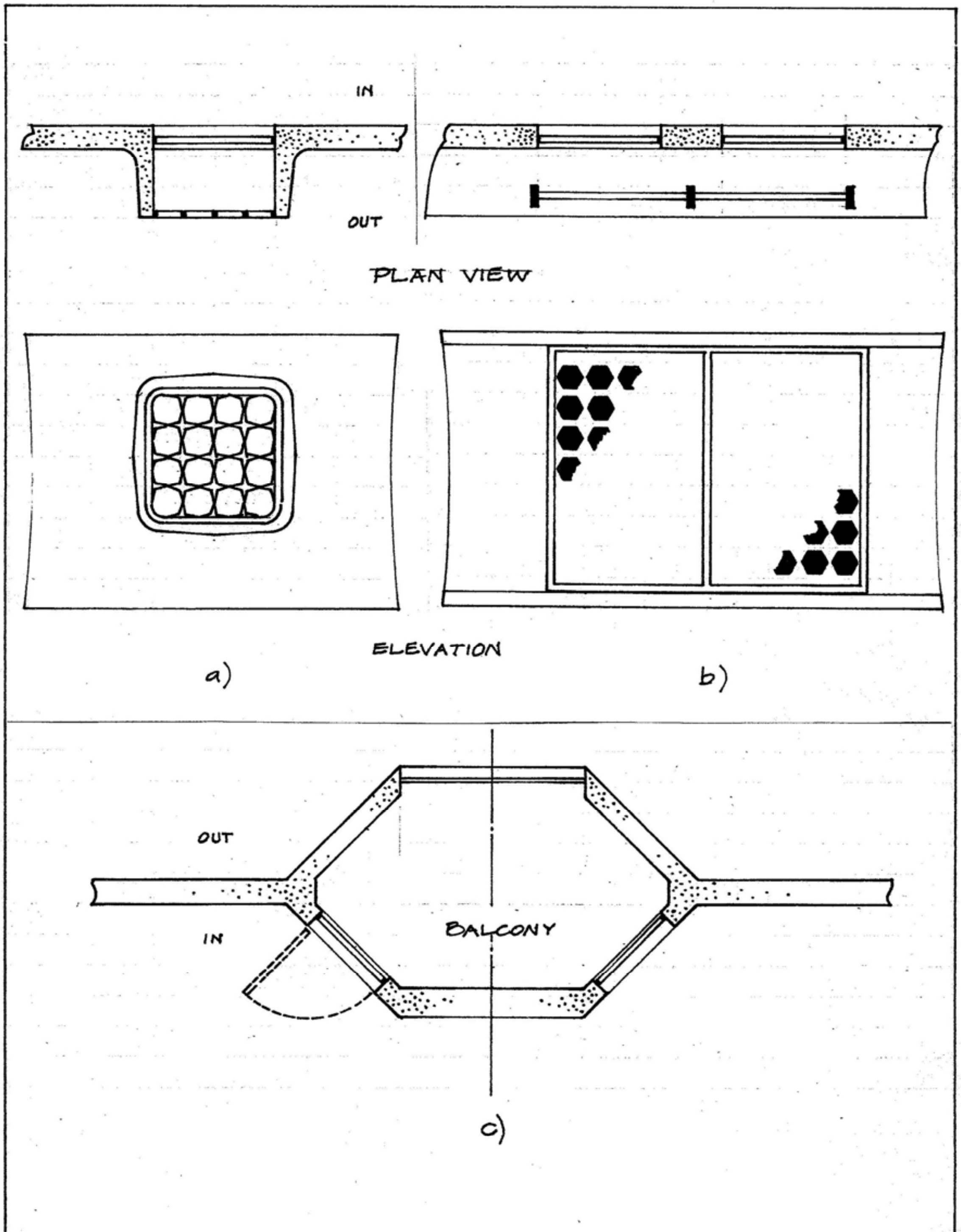


FIGURE 5.5 OPENING PROTECTION METHODS

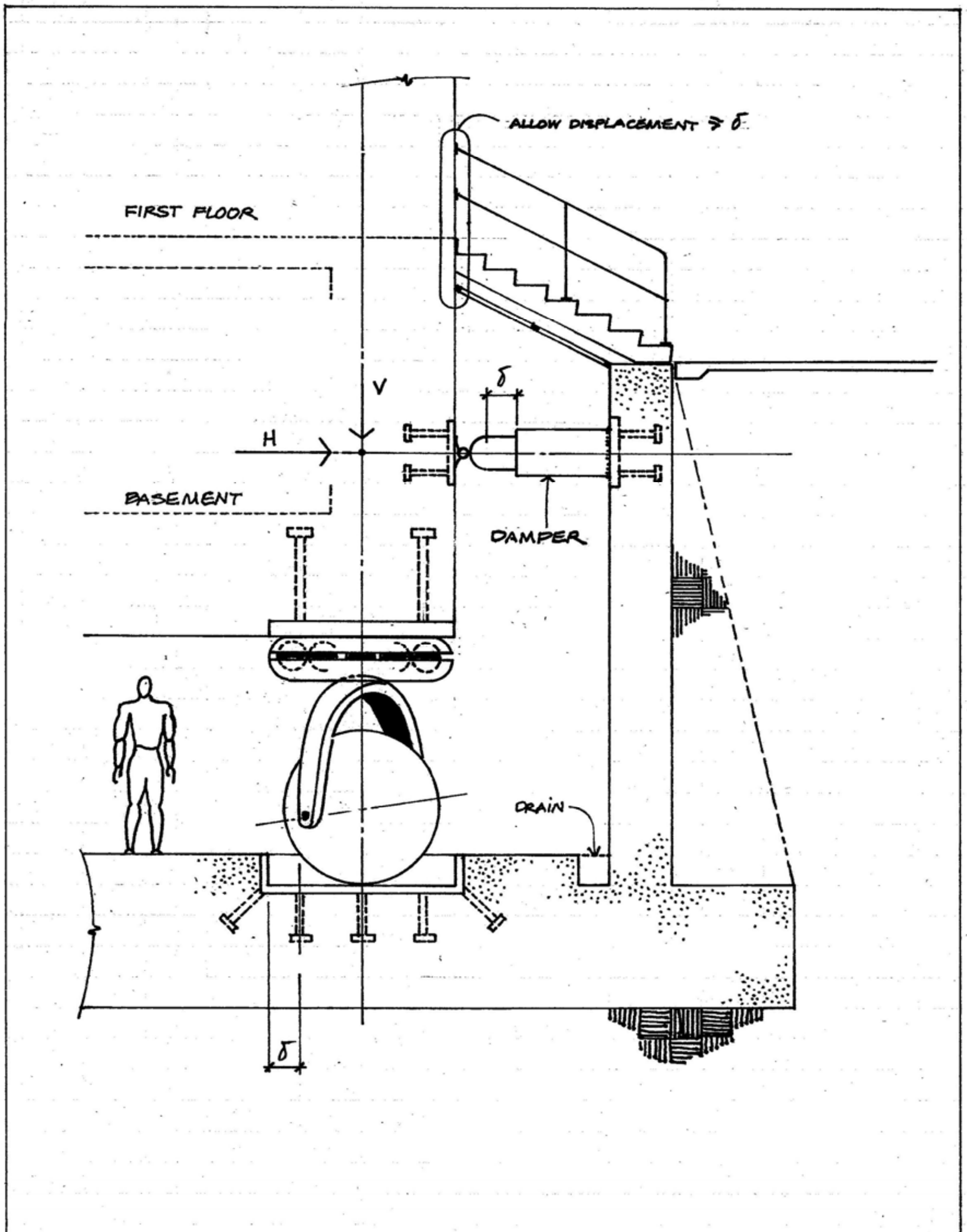


FIGURE 5.6 VISION OF THE FUTURE

APPENDIX A

The following is a verbatim reproduction of the foreword of a certain lecture delivered in 1999 by the author as the then dreaded Y2K approached. The title of such lecture was *The Notion of the Result.* Here is a transcription of such wording:

"The anticipated notion of the result is one of those highly desirable abilities of the design engineer. Sooner or later we all reach that level of anticipation as brought up by experience and the constant repetition and comparison of results.

The same way that the experienced design engineer knows ahead of time that a one-way concrete slab with a span of 10 feet and subject to roof loads could be solved with a thickness of 4 inches and main reinforcement of #4 @ 12 in. on centers, in the same way he could also anticipate the necessary slenderness of a high rise building subject to hurricane winds or seismic motions. Furthermore, in one first approximation he should be able to decide which column spacing is best, as well as the appropriate beam size for the architectural solution in front of him. Naturally, with the availability of nowadays computers, the novice could make a few trial and errors here and there until reaching the point of correct *convergence.*

After such a reasoning it becomes evident that *the notion of the result* is not only quite handy but desirable ability for the design engineer to have, specially due to the eternal quandary of design so as to have the ability to before-hand assume sizes which would be closest to those which can produce the stresses better suitable to the used material.

Conclusively, such an ability is not only useable, handy and convenient to have, but more than that, it indeed brings a sense of certainty, safety and accuracy when it comes to the suitability of the results."

APPENDIX B

With computers everywhere and for every possible and imaginable use, it would be now hard to imagine those bygone days when engineers did every piece of calculation by hand. Back in 28the early 1960's only the government and very large offices could afford to have a "computer room", in most cases a 14 x 30' room with an elevated floor to allow the necessary extensive wiring. The room was filled with enormous and slow CPU's fed by punched cards. Because of all the heat generated by such machines operating with antiquated electronic circuitry, those rooms had to be equipped with very efficient air conditioning systems able to maintain a steady 72 degree temperature.

We engineers started to learn the jargon and became aware of that new "machine language" and the need to create programs directed to solve our problems in a particular sequence so to avoid the machine to fall into the dreaded so-called "loop". Soon, commercial engineering programs (filled with bugs I must add) started to appear in the market. Along with the *finite element method* fever, smart programs such as STRUDL also emerged taking a giant step ahead of the machines.

New developments in electronics enabled the industry to start reducing the size and at the same time to increase the storage and processing capacity of computers, to the point we are at today where that above described 14 x 30' computer room has been reduced to a 1.5" x 10" x 15" *laptop.*

The enclosed example of hand computations (10 pp. out of 32) as created in May 1974, are indeed a relic from the past when engineers did everything by hand and computers were a somewhat distant possibility in process of development. Such a venerable example consists of manual computations for the design of a five story medical office building. Floor framings consisted of an 8.5 in. thick post-tensioned flat plate carried by concrete columns. Lateral loads due to wind pressure were solved by using reinforced concrete shear walls naturally provided by the necessary stairwells and elevator tower.

As result of the poor and swampy soils often found in South Florida, in this case history all columns and shear walls were supported by a pile foundation system. All design followed the requirements of the then enforced and original South Florida Building Code.

1

PROFESSIONAL OFFICE CENTER

MAY 6, 1974.

GENERAL DESCRIPTION:

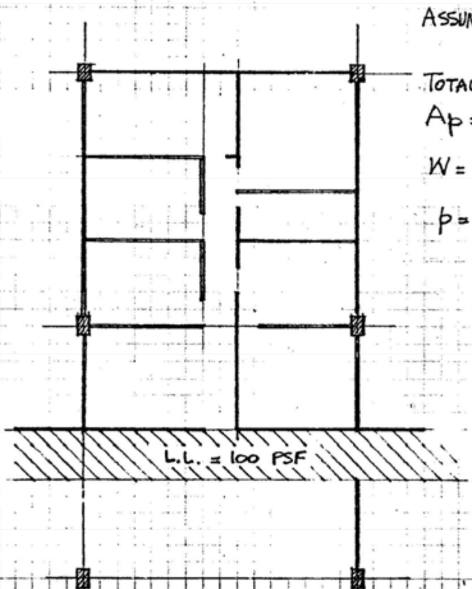
FIVE STORY BUILDING, BASE DIMENSIONS: 90' x 160'
OCCUPANCY: MEDICAL OFFICES (OUTPATIENT)
BAY SIZE: 30' x 32' (TYPICAL)
CONCRETE COLUMNS, SHEAR WALLS, SLABS, STAIR & ELEVATOR CORES.

STRUCTURAL CONCEPT:

POST-TENSIONED FLAT PLATE SUPPORTED BY RECTANGULAR TIED COLS.
LATERAL LOADS DUE TO WIND RESISTED ENTIRELY BY SHEAR WALLS.

LOADING CRITERIA:

PARTITION LOADS



ASSUMED PARTITION LAY-OUT

$A = 30 \times 32 = 960$ SF.

TOTAL DRYWALL PARTITION LENGTH; $L_p = 180$ LF.

$A_p = 180 \times 10 = 1800$ S.F.

$W = 1800 \times 6 = 10{,}800$ #

$p = \frac{10{,}800}{960} = 11.25$ SAY 12 PSF

[USE 20 PSF TO MEET CODE 2301.2(e)]

LOADING CRITERIA:

LIVE LOADS:

OFFICES:	50	PSF
CORRIDORS:	80	"
ROOFS:	30	"
STAIRS:	80	"

DEAD LOADS:

8" SLAB: < 100 PSF (150 PCF CONC; 97 PSF (145 PCF CONC

PARTITIONS:	20	PSF
CEILINGS:	5	"
CARPETING:	1	"
MECHANICAL:	2	"
ROOFING:	10	"

ADJUSTMENT IN LIVE LOAD TO COMPENSATE FOR HIGHER L.L. IN CORRIDORS:

$A = 92 \times 161.50' = 14858 \quad \times 50 = 742{,}900$ #

$A' = 1776 \quad \times 30 = 53{,}280$

$796{,}180$

$p' = \frac{796180}{} = 53.6$ PSF

MAY 9, 1974. 6

$$f = -339 \pm \frac{12 \times 6{,}330}{128} = -339 \pm 593 = \begin{array}{l} +254 < 380 \\ -932 < 1200 \text{ PSI.} \end{array}$$

@ 2 : M = 256 KIP-FT (8.53 K-FT/FT)

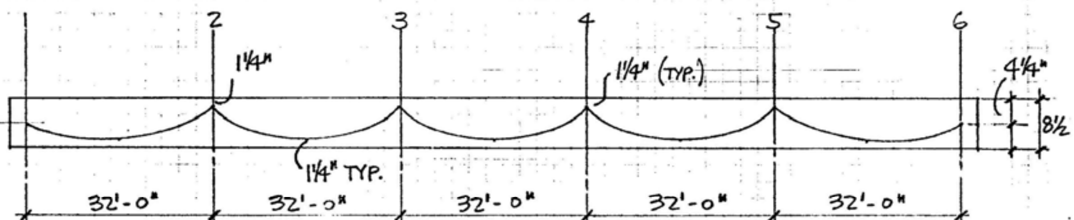
$$f = -339 \pm \frac{12 \times 8530}{128} = -339 \pm 800 = \begin{array}{l} +461 > 6\sqrt{f'_c} = 380 \text{ PSI.} \\ -1139 \text{ PSI.} \end{array}$$

ABOVE STRESS IS TOO HIGH, SLAB THICKNESS SHALL BE INCREASED TO 8½".

BASED ON 145 PCF, SLAB WEIGHT IS : $8.5/12 \times 145 = 102$ PSF, WHICH WILL NOT AFFECT LOAD ANALYSIS MADE ON SHTS 1, 2 & 3.

GOING BACK TO TENDON PROFILES OF SHT. # 3 :

1- LONGITUDINAL STRIPS (OTHER THAN ELEVATOR CORE) - FLOOR -



Wpr.	90	90	90	90	90	PSF
Mpr.	11.50	11.40	11.40	11.40	11.50	FT-KIPS
a	4½"	6"	6"	6"	4½"	IN.
F	31.14	23.04	23.04	23.04	31.14	KIPS/FT.
F/A	305	226	226	226	305	PSI.

$A = 12 \times 8.50 = 102$ in^2

MAY 12, 1974

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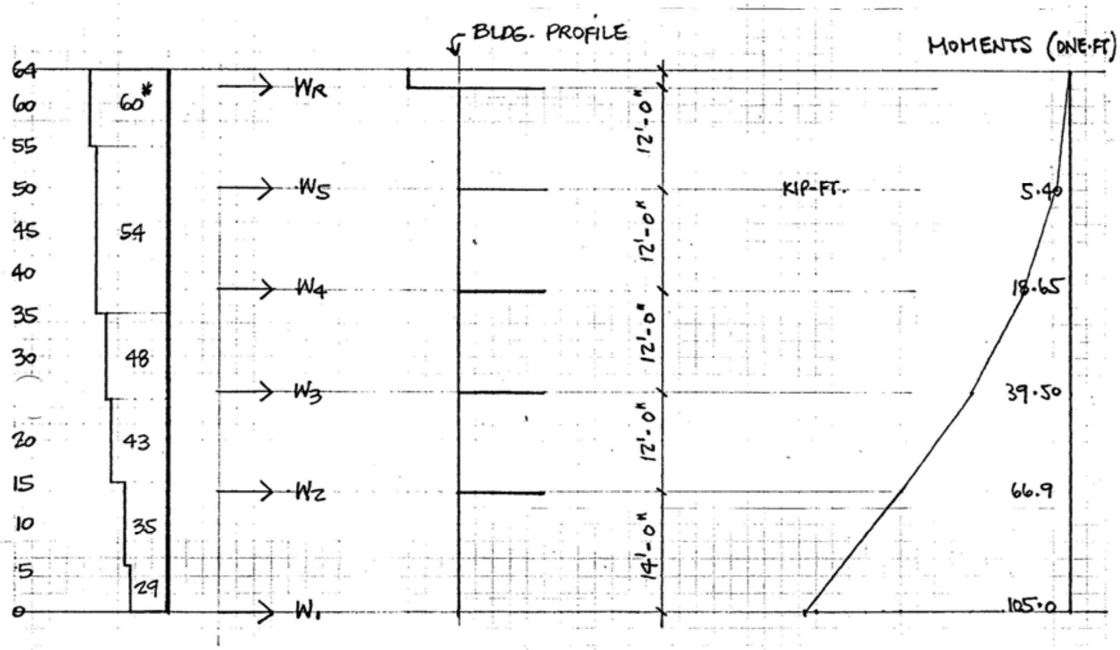
WIND ANALYSIS :

So. FLORIDA BUILDING CODE

WIND CONSIDERED ACTING EAST-WEST.

OVERALL BUILDING LENGTH : L = 172'-2"

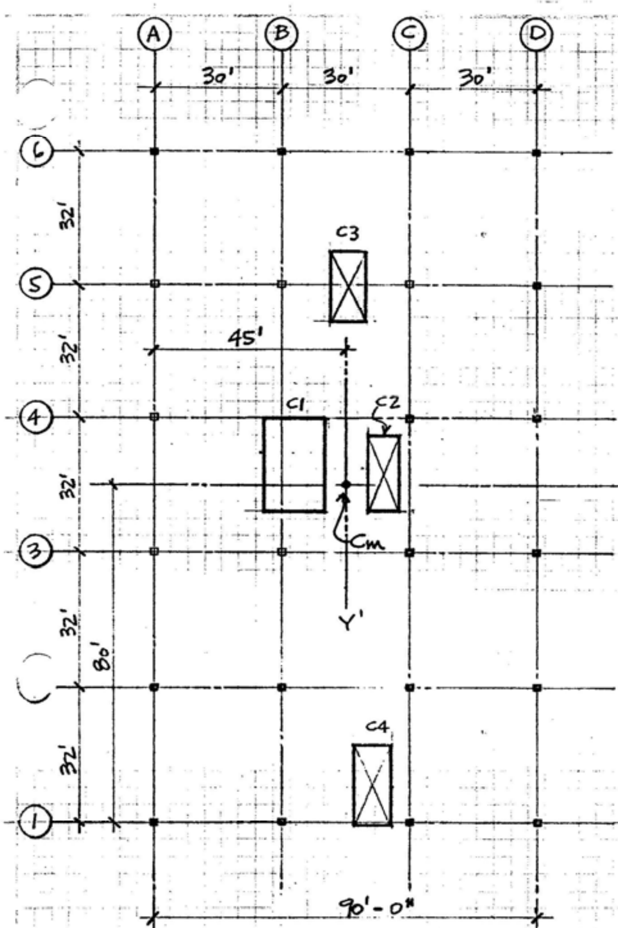
OVERALL BUILDING HEIGHT : H = 63'-6"



LOADS FOR 1 FT OF BUILDING :

		TOTAL LATERAL LOADS : (x 172.16) =
$W_1 = (7 \times 29) + (2 \times 6)$	= 215 #	37.01 K
$W_2 = (13 \times 35) + (5 \times 8)$	= 495 #	85.22 K
$W_3 = (12 \times 43) + (7 \times 5)$	= 551 #	94.86 K
$W_4 = (12 \times 48) + (9 \times 6)$	= 630 #	108.46 K
$W_5 = (12 \times 54) + (1 \times 6)$	= 654 #	112.59 K
$W_R = (7.50 \times 60)$	= 450 #	77.47 K
		V = 515.61 K

MAY 13, 1974. 22



SHEAR WALL ANALYSIS
(WITH CONSIDERATION OF TORSION)

"X-X" AXIS RUNNING EAST-WEST.

ALL SHEAR WALLS 8" THICK.

CORE SIZES:

C1	16' x 23"	(OVERALL)
C2	7' x 19'	
C3	9' x 17'	
C4	9' x 17'	

Cm = CENTER OF MASS.

AS INDICATED ON SHT. #1, ONLY SHEAR WALLS ARE CONSIDERED FOR THE PURPOSE OF WIND RESISTING ACTION.

CASE 1:

WIND ACTING EAST-WEST (PARALEL TO AXIS X).

BASE SHEAR : $V_u = 670^K$

SHEAR WALL DATA (X DIRECTION)

S/W	d(IN)	b(IN)	A(IN²)	H(in)	I(IN⁴)	Δ_m	Δ_v	Δ_t	K	D(%)	SHEAR (K)
C4N	108	8	864	144	839,800	0.09"	0.16"	0.25"	4.0	0.31	208
C2S	84	8	672	144	395,100	0.20"	0.21"	0.41"	2.4	0.18	127
C2N	84	8	672	144	395,100	0.20"	0.21"	0.41"	2.4	0.18	127
C3N	108	8	864	144	839,800	0.09"	0.16"	0.25"	4.0	0.31	208

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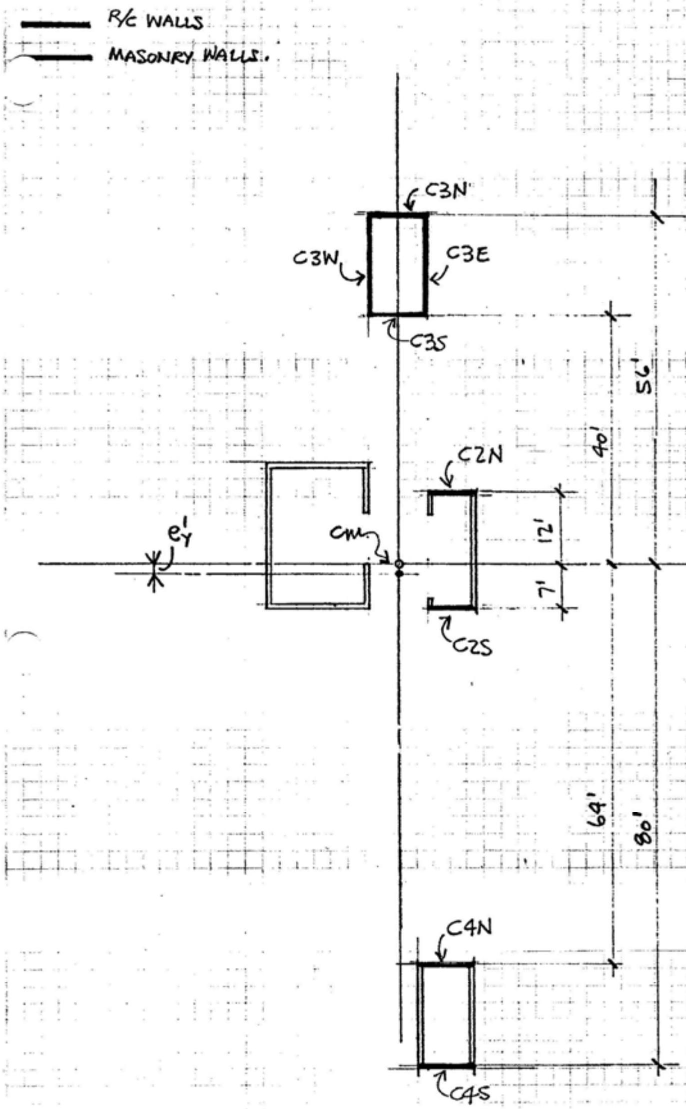

TORSION :

$e_y = \frac{37}{1213} = 0.0305' \sim 3/8"$

$e_x = 0$

$e_y' = 1.48' \sim 1'\text{-}5\frac{3}{4}"$

$\Sigma K = 2.94$

$M_t = 670 \times 1.48 = 991.6$ K-FT

SHEAR REDISTRIBUTION DUE TO TORSIONAL MOMENT :

$P_{4N} = 1.0$

$P_{2S} = P_{4N} \frac{0.18}{0.31} \times \frac{5.52}{62.52} = .05 P_{4N}$

$P_{2N} = P_{4N} \frac{0.18}{0.31} \times \frac{13.48}{62.52} = .125 P_{4N}$

$P_{3N} = P_{4N} \frac{0.31}{0.31} \times \frac{57.48}{62.52} = .92 P_{4N}$

$991.6 = P_{4N} \times 62.52 +$
$5.52 \times P_{2S} +$
$13.48 P_{2N} +$
$57.48 P_{3N} =$

$991.6 = 62.52 P_{4N} + (5.52 \times .05) P_{4N} + (13.48 \times .125) P_{4N} + (57.48 \times .92) P_{4N}$
$= (62.52 + 0.276 + 1.685 + 52.882) P_{4N} = 117.36 P_{4N}$

$P_{4N} = \frac{991.6}{...} = 8.44^{K}$

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SHEAR WALL DATA (X DIRECTION).

S/W	d (IN)	b (IN)	A (IN^2)	H (IN)	I (IN^4)	Δ_m (IN)	Δ_v (IN)	Δ_t (IN)	K	DISTR.	SHEAR (KIPS)
C4S	108	8	864	180	839,800	0.19	0.21	0.40	2.5	0.19	127.30
C4N	108	8	864	180	839,800	0.19	0.21	0.40	2.5	0.19	127.30
C2S	84	8	672	180	395,100	0.41	0.27	0.68	1.47	0.12	80.40
C2N	84	8	672	180	395,100	0.41	0.27	0.68	1.47	0.12	80.40
C3S	108	8	864	180	839,800	0.19	0.21	0.40	2.5	0.19	127.30
C3N	108	8	864	180	839,800	0.19	0.21	0.40	2.5	0.19	127.30
								$\Sigma =$	12.94	1.00	670.00

$(2.50 \times 80) + (2.50 \times 64) + (1.47 \times 7) = \quad + 370.29$

$(1.47 \times 12) + (2.50 \times 40) + (2.50 \times 56) = \quad \underline{- 257.64}$

$+ 112.65 \quad (M_K)$

$e'_y = \dfrac{112.65}{12.94} = 8.71^{FT}$ (SOUTH FROM CM).

$M_t = 670 \times 8.71 = 5835.70$ KIP-FT (TORSIONAL MOMENT).

VALUES REFERRED TO C4S:

$P_{4N} = P_{4S} \dfrac{0.19}{0.19} \times \dfrac{55.29}{71.29} = 0.776 \, P_{4S}$

$P_{2S} = P_{4S} \dfrac{0.12}{0.19} \times \dfrac{1.71}{71.29} = 0.015 \, P_{4S}$

$P_{2N} = P_{4S} \dfrac{.12}{.19} \times \dfrac{20.71}{71.29} = 0.183 \, P_{4S}$

$P_{3S} = P_{4S} \dfrac{.19}{.19} \times \dfrac{48.71}{71.29} = 0.683 \, P_{4S}$

$P_{3N} = P_{4S} \dfrac{.19}{.19} \times \dfrac{64.71}{71.29} = 0.908 \, P_{4S}$

$M_t = 5835.70 = 71.29 P_{4S} + 55.29 P_{4N} + 1.71 P_{2S} + 20.71 P_{2N} + 48.71 P_{3S}$

$+ 64.71 P \quad = 71.29 P_{4S} + 55.29 \times .776 \times P_{4S} + 1.71 \times .015 P_{4S} +$

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$M_t = 5835.70 = P_{4S}(71.29 + 42.91 + 0.03 + 3.79 + 33.27 + 65.62)$

$= 216.91\ P_{4S}$

$P_{4S} = \frac{5835.70}{216.91} = 26.9^K$

$P_{4N} = .776 \times 26.9 = 20.87^K$

$P_{2S} = .015 \times 26.9 = 0.40^K$

$P_{2N} = .183 \times 26.9 = 4.92^K$

$P_{3S} = .683 \times 26.9 = 18.37^K$

$P_{3N} = .908 \times 26.9 = 24.43^K$

REDISTRIBUTED SHEARS →

S/W	SHEAR (K)	%
C4S	100.40	15
C4N	106.43	16
C2S	80.80	12
C2N	85.32	13
C3S	145.67	21
C3N	151.73	23
	670.35	100

STAIR WALLS (SW-1)

$M_u^G = .23 \times 23{,}500 = 5405$ KIP-FT. (GND. FLOOR)

$f'_c = 4000$ PSI. $f_y = 60{,}000$ PSI.

$b = 8''$ $h_w = 108''$ $d = 96''$

$F = 6.14 = \frac{M_u}{K_u}$ $K_u = \frac{5405}{6.14} = 880$ $a_u = 3.71$

$A_s = \frac{5405}{3.71 \times 96} = \frac{5405}{356.16} = 15.18\ in^2$ (10 # 11)

$M_u^2 = .23 \times 14{,}973 = 3444$ KIP-FT. (2^{ND} FLOOR)

$A_s = \frac{3444}{356} = 9.67\ in^2$ (8 # 10)

$M_u^3 = .23 \times 8{,}840 = 2033$ KIP-FT. (3^{RD} FLOOR)

$A_s = \frac{2033}{407} = 5.0\ in^2$ (6 # 8) $K_u = 331$ $a_u = 4.24$

$M_u^5 = .23 \times 1208.56 = 278$ KIP-FT. (5^{TH} FLOOR) $K_u = 46$

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ELEVATOR WALLS (SW-2)

$b = 8''$ $\ell_w = 84''$ $d = 72''$ $F = \frac{8 \times 72^2}{12000} = 3.46 = \frac{M_u}{K_n}$

$M_u^G = 0.13 \times 23{,}500 = 3055$ KIP-FT. (GND. FLOOR).

$K_u = \frac{3055}{3.46} = 883$

$A_s = \frac{3055}{3.71 \times 72} = \frac{3055}{267} = 11.43 \text{ in}^2$ (8 # 11)

$M_u^2 = 0.13 \times 14{,}973 = 1946.5$ KIP-FT. (2ND FLOOR)

$K_u = \frac{1947}{3.46} = 563$ $a_u = 4.04$

$A_s = \frac{1947}{291} = 6.69 \text{ in}^2$ (6 # 10)

$M_u^3 = 0.13 \times 8{,}840 = 1149.20$ KIP-FT. (3RD FLOOR).

$K_u = \frac{1150}{3.46} = 333$ $a_u = 4.23$

$A_s = \frac{1150}{305} = 3.77 \text{ in}^2$ (4 # 9)

$M_u^4 = 0.13 \times 4174 = 543$ KIP-FT. (4TH FLOOR).

$K_u = \frac{543}{3.46} = 157$ $a_u = 4.38$

$A_s = \frac{543}{315} = 1.72 \text{ in}^2$ (2 # 8)

$M_u^5 = 0.13 \times 1209 = 157$ KIP-FT. (5TH FLOOR).

$A_s = 0.50 \text{ in}^2$ (2 # 5)

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TENSION ON PILES:

$T_{max} = (5.90 \times 18) \div 4 = 26.6$ TON. O.K.

DESIGN OF PILE CAP:

$-M = (189 \times 4.50) + (138.6 \times 1.50) = 850.5 + 207.9 = 1059$ KIP-FT.

$b = 6'-0" = 72"$ $t = 36"$ $d = 30"$

$f'_c = 3000$ PSI. $f_y = 60{,}000$ PSI.

$A_s = \frac{1059}{53} = 19.98$ in^2 13 #11 (LONGITUDINAL) (B

10 #9 (T

$M_t = \frac{189}{2} \times \frac{1.50}{3} = 47.25$ KIP-FT/FT.

$A_s = \frac{48}{53} = 0.91$ in^2

$A_{s\,min} = .0020 \times 12" \times 36" = 0.86$

$\Big\}$ #8 @ 10" O.C. (TRANSV.) ✓

STIRRUPS #3 @ 15" O.C.

SW-2 (ELEVATOR WALLS)

$M' = \frac{0.13 \times 23{,}500}{1.30} = 2{,}350$ KIP-FT. (@ GND. FL. LEVEL)

$M_u = 3055$ KIP-FT. $P = 288^K$

$M" = 2{,}350 + 403 = 2{,}753$ KIP-FT. (@ ELEV. PIT LEVEL).

$P" = 288 + 42 = 330^K$

ASSUMED FOOTING SIZE: 6' x 18'

$A_f = 108$ FT^2 $I_f = 2916$ FT^4

$f_{max} = 330 + 2753 \times 9 = 3.06 + 8.50 = 11.56$ K/SF.

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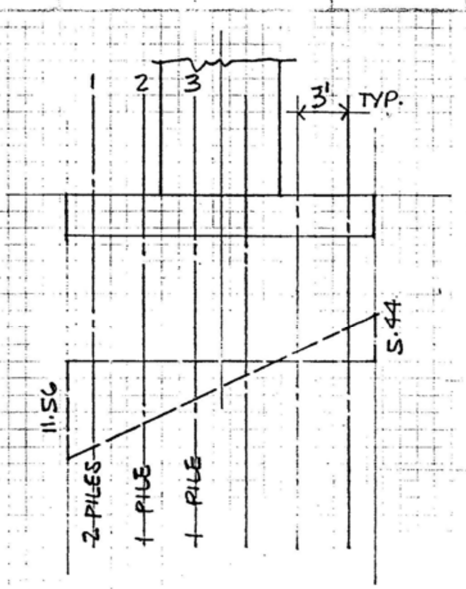

$f_{min} = 5.44$ KSF

$F_1 = 10 \times 18 = 180^K$ (2 P)

$F_2 = 7.4 \times 18 = 133^K$ (1 P)

MAX. TENSION ON PILES:

$T_{max} = (4.0 \times 18) \div 4 = 18.0$ TON.

DESIGN OF PILE CAP:

$-M = (180 \times 4) + (133 \times 1) =$

$= 720 + 133 = 853$ KIP-FT.

$b = 72''$ $t = 36''$ $d = 30''$

$f'_c = 3000$ PSI. $f_y = 60{,}000$ PSI.

$A_s = \frac{853}{53} = 16.10 \text{ in}^2$ 13 # 10 (LONGITUDINAL)(

7 # 10 (T)

$M_t = \frac{180 \times 1.50}{6} = 45$ KIP-FT/FT.

$A_s = \frac{45}{53} = 0.85 \text{ in}^2 < 0.86 \text{ in}^2$ # 8 @ 11" o.c. (TRANSV.)

STIRRUPS # 3 @ 15" o.c.

END