The Towers, Fire- Induced Collapse and the Building Codes

Summary;
A preliminary interpretation of the evidence in the analysis of the cause of the large loss of life and collapse of the World Trade Center Towers & and recommended code changes to mitigate the chance of a recurrence, in high-rise office buildings.

July 17, 2003 Draft #13

By: Arthur Scheuerman, Battalion Chief FDNY (Ret.), Former Instructor Nassau County Fire Training Academy and high-rise Fire Safety Director NYC.- E-Mail, acaj1@aol.com.

There is increasing evidence that the fire was the main cause of the collapse of the WTC Towers and there were serious design, construction and fire protection deficiencies that made the building unusually susceptible to fire. This report is my contribution to determining the causes of the large life loss and building collapse in the Towers, in hopes of developing design parameters and code changes to mitigate such hazards.

New York City has been, over the past hundred years, a leader in high-rise code development and has been copied by many other cities. When the new 1968 NY City Building Code came out, I was in the Fire Dept. studying the 1938 code. We naturally thought this “new” ’68 code was an improvement over the ‘old’ code and were looking forward to studying it as advancement in construction practices. Gradually, as we waded through the increasing complexities, we realized that several requirements had been ‘left out’ of the new regulations and others weakened. The regulation requiring hardened, smoke proof, ventilated stairways (called ‘fire towers’) in high-rise buildings was missing along with the requirement for a fire hose impact test, for fire doors, walls and floors, after they had undergone the furnace test to assure structural stability and the prevention of fire and smoke passage out of the room or area . Various other changes were made reducing the fire resistance for walls and floors and increasing the area between fire separation barriers.

The story we heard from the building industry was that the ‘old’ regulations were unnecessary and the older buildings were “overbuilt” because of ‘old time’ engineer’s ignorance of the real capabilities of steel frame construction. The previous builders were putting too much weight in their buildings using heavy steel and concrete and wasting time and money in the construction process.

Well, if this was ever true, to begin with, it appears the pendulum has swung too far in the opposite direction towards relaxatation of the regulations, and using cheaper, lighter weight materials, for steel frame, high-rise buildings. The city is presently in the process of upgrading the Building Codes in the wake or the World Trade Center disaster and this essay is my perspective, as a retired NYC Fire Chief, in furtherance of that process.

Every large building can be expected to be subjected to one or more fully developed fires during its lifetime and fire is one of the main reasons Building Codes were developed. HERA engineer Charles Clifton speaking of the likelihood of fire in a building says… “In an unsprinklered building, the fully developed fire condition is at least 5 times more likely to occur over a 50 year timeframe than is the severe earthquake, which is more likely than any other structurally damaging event." (Clifton DCB #71) The way the building performs in protecting the occupants from fire and smoke and maintaining its integrity during a fire is largely a function of its design and construction and of compliance with realistic building codes. Not all high-rise buildings will be hit by planes, but experience has shown they will be exposed to fire. However, now all three of the tallest buildings in this city have been hit by planes and we are compelled to consider this hazard also, when building super tall buildings.

The most probable cause and mechanism of collapse of the Towers is presently under contention. There are two main theories of collapse, one based on core column damage from the aircraft collision and subsequent core column failure from the heat of the fire, and the other from the effects of the fire on the steel bar joist, floor trusses. Due to the strong emotions evoked by the tragedy, a bias naturally affects the perception of the causes of the collapse. Each theory stems from the position an analyst is in. Those in the building industry naturally feel their work is the best possible and gravitate to defending their design and construction work and tend to emphasize the aircraft as causing the most damage and being mostly responsible for the collapse. Some are not expecting building deficiencies and are, therefore not looking for, and are, naturally, less likely to find any design or construction flaws.

For example, in the Silverstein study, in the search for the collapse origin, they did not allow the computerized model to consider floor failure as a possibility. "A consultant for the insurance companies said the computer analysis assumed that the floors stayed intact until the collapse." This model naturally predicted column failure as the cause. "The assertion that the floor design played no role was in some ways foreordained. The mechanism they predict is really the only one they can predict, given the model they use.", said Dr. Osteraas, director of civil engineering at Exponent Failure Analysis (NYT, Glanz, 10/22/02)) Predictably column damage from the aircraft impact and heat from the fire was blamed for the collapse.

To find floor truss failure as a progressive collapse cause would indicate that the fire was responsible and, by code and design, high-rise buildings are required to be protected against collapse from fire. Therefore we see that the aircraft is found responsible for column destruction and the removal of fireproofing on the remaining columns, thus de-emphasizing the documented deficiencies in fireproofing throughout the buildings. The regular inspection of the fireproofing is emphasized, while the installation, repair and maintenance deficiencies, discovered during these inspections, is downplayed.

Tower two (the South tower) is determined to have collapsed because the aircraft 'damaged and compromised the corner of the core of the building.' There is no mention of the floor trusses possibly buckling due to thermal elongation and bowing and resulting compression and forming a deflected shape, plastic hinge or sagging further, due to weakening from the heat of the fire, thus producing tensile forces, pulling in the entire perimeter bearing wall along the East side. These are all convenient omissions which avoid taking cognizance of and responsibility for the loss and assuring correction of these design or construction deficiencies. What we need is a scientific determination of the actual mechanisms responsible for the complete collapse of these huge, supposedly "fire-resistive" buildings.

Many engineers writing reports on the collapse use phrases such as ‘magnificent buildings’ or ‘superb engineering’ when referring to the Towers. Those of us in the fire service, realize that there are lessons to be learned about the collapse and are interested in finding the actual weaknesses in design or construction which can be used in the prediction and prevention of fire induced collapse and life loss, to avoid a repetition. We wouldn’t begin a report, which possibly may uncover major design faults or construction deficiencies, with phrases like ‘that magnificent building’ and ‘superbly engineered’. A statement like that would surly mean that we were not ready to find any weaknesses in a buildings design or construction. We naturally put less emphasis on the effect that the planes had on the buildings, in hopes of finding the weaknesses in fire protection and countering the tendency by the builders to blame the plane thus attributing most of the damage and subsequent collapse to the collision,

If I used the words “superbly engineered”, I would have to add; - to build a majestic structure, to attract and house the most tenants, in the smallest footprint at the least cost, to make the most money, with little regard for, and little understanding of fire safety design, construction and maintenance. The American Society of Civil Engineers even presented the WTC with a national award for outstanding achievement in 1971, and then they were put in charge of the investigation of the collapse. How dispassionate could the ASCE be in its analysis of the collapse after giving the building an award?

Some of the design weaknesses and construction deficiencies contributing to the vulnerability of the Trade Center buildings were, to my mind endemic in the industry, the product of many years of enthusiasm by owner/builders in the City, to maximize profits by producing grand, spacious office occupancies with spectacular views and elaborate interiors to attract tenants and increase rental income. Some of the means used to accomplish these goals were in direct competition with fire safety regulations. Caught in the competition to attract tenants and reduce costs they worked, apparently through the political system, to reduce the requirements imposed by the previously effective, NY City Building Codes compiled in the 1938 version. The resulting, relaxed, 1968 code revision, and later special interest revisions, coupled with the fact that the Port Authority as a two-state agency was not required to follow even these weakened regulations, allowed the design deficiency and construction flaws in the Twin Towers, which are becoming evident after this tragic disaster.

The deterioration in mandated fire safety with the prospect of large life loss and possible collapse, which may be surfacing, in these third generation high-rise buildings, developed, over the years, from efforts towards deregulation stemming from this competition, for tenants and to reduce costs and maximize profit in these commercial buildings. The most economical and rapidly constructed buildings had a financial reward for the developer. The absence of a strongly worded, unambiguous Building Code allowed excessive skimping on fire safety by developers in order to compete and remain in business. Glitz and glamour and expansive cityscape views took precedence over safety. The result was the lightweight, ‘core constructed’ high-rise building with an open floor plan, of which, the Twin Towers design, was the most extreme example. The City is adopting the International Building Code (IBC) and re viewing it for necessary changes.

Increasing Rentable Areas

This fervor by owner/builders to attract tenants by increasing rentable areas and improving tenant flexibility and creating ambience by opening up the floor space, providing grand views and utilizing every square foot of area within the building plan, has led to the following excesses evident in these newer, open floor plan, 'core constructed' high-rise office buildings. **One** - The City Code allows gigantic height and building area possibilities, as evidenced in Table 4-1 and 4-2 of Chapter 27 which places 'no limits' on heights and areas in certain types of buildings. Note #1-This allowed a Port Authority public relations expert to suggest, as a marketing device to attract tenants, that they build the tallest building in the world with maximum square footage. The PA management talked the architect into it, after he protested that "You can't build buildings taller than eighty stories." (Gillespie, p.47) **Two**. – With the increased floor areas the maintenance of exterior views and light for all tenants within these enlarged floor areas, required elimination of interior walls, - walls that previously confined fires, to areas that were manageable by firefighters, (2500-5000 sq. ft.), on the upper floors of a high-rise building. **Three**. - Brought about removal the enclosed, fire resistant, interior corridors previously used by firefighters to safely approach and quickly extinguish interior fires. If you don't have private offices you don't need corridors to get to them. These corridors provided a barrier to fire spread, assisted confinement, protected escape routs and stairways and added to the stability of the building. **Four**. – The grouping, of all the necessary construction which would obstruct light and cityscape views, to a core area, necessitated interpreting the building code definition of 'remote' to mean allowing grouping of stairwells as close as 15 feet apart, so that, in the North Tower, all the stairway escape routes and elevators were close together and were incapacitated by the impact of the plane or by the fuel-air explosion, trapping 500 people above the fire. **Five** - The removal of interior columns. This planned, column deletion, in the large area building necessitated long-span construction, which was satisfied in the WTC Towers by using lightweight, open-web, steel bar joists which are susceptible to early failure during fires and subject to sequential collapse, exposing long stretches of columns to pull-in forces and now implicated in the progressive, building collapse. **Six.** – Elimination of the code requirement for hardened fire and smoke proof stairways (called 'fire towers' - which, in effect, put two fire doors between the occupancy and the stairway, and provided a chimney through the roof to draw off any smoke or fire before it reached the stairway). This saved space and cost by eliminating the enclosed vestibule and its smoke removal shaft. These and other weaknesses in the code along with the design and construction deficiencies contributed in, some measure, to the exposure of large numbers of people in the WTC to the peril of fire and collapse.

Large Open Areas

Large open areas, containing combustibles, within buildings, are a nightmare for firefighters because of the possibility of spread of fire, throughout the space and the possible large volume of fire. This fire growth in large open areas is exacerbated in high-rise structures when elevators which have proven unreliable at fires, and stairways and standpipes must be used by responding firefighters, delaying the operation of hose streams and complicating rescue. As Deputy Chief Dunn pointed out "firefighters cannot extinguish a fire in a 20- or 30-thousand-square-foot open floor area in a high rise building." (For an article on the operational problems related to design and construction, at high rise fires; see Chief Vincent Dunn's article in Fire Engineering magazine December'95.)

The difficulty or impossibility in extinguishing such large, open area fires when they are fully developed and extend throughout an interior space in a high-rise, arises, I believe, because, as the fire in one section is extinguished and the hose streams are repositioned to attack another area, the fire re-ignites, in the previous section by convected and radiated heat from the freely burning section and from hot areas shielded from the water streams. The convected and radiated heat becomes an impossible barrier to advancement of the limited number of hose lines that can be fielded in the upper areas of such buildings. This hazardous situation occurs even in well-ventilated areas and fire-suppression of well developed fires in such large open areas, within buildings, often requires, in my estimation, the cooling of all spaces at once, an effect, which because of the limitations on interior hose line operations in the upper stories of high-rise buildings, can only be accomplished by the water spray of a well supplied sprinkler system. I recommend that NIST determine the area of fire that can be controlled by a well supplied sprinkler system. HERA structural engineer Charles Clifton recommends having the most reliable suppression system practicable when the individual fire area exceeds a threshold of 1500 m. square. (approximately 5000 sq. ft.). This would apply to all buildings. High-rise buildings should have a reliable, suppression system throughout all rooms and areas of the building. This is because of the higher probability of fire, because of the larger area and numbers of people; greater problems with rescue and fire extinguishment and the resultant higher hazards, of life loss and property damage possible.

Chief of Department John O'Hagan writing in "High-Rise Fire and Life Safety" in 1977 stated:

"The main problem associated with the protection of life from fires in high-rise buildings is the limitation of the size of the fire. Public fire protection services can usually contain a fire in an elevated portion of the building if the fire area is limited to 5000 square feet of less. If the horizontal spread of the fire exceeds this limit, the heat developed will [often prevent approach, coverage and extinguishment by hand held hose lines and] also increase the risk of vertical spread. If the building contains vertical arteries, then a tragedy can be expected." (p245, 246), (My bracketed […] insert).

Fire Containment – Breaches in Fire Resistant Floors, Walls

. Any openings in such large area floors, such as un-enclosed access stairs, poke through holes or other breaches in the fire barrier floor could be perilous due to the time and difficulty in extinguishing a fire engulfing an entire floor, and the almost certainty of fire and smoke extension to the floor or floors above. When you have a fully developed fire in such a large area of a building and there are open access stairs, escalators or other openings in containment floors, for the fire to extend to the floors above, control of fire extension may not be feasible unless control can first rapidly be accomplished on the lowest fire floor. Even then the fire can extend up through such openings to the upper floors faster than it can be extinguished. A fire on two floors in this type of structure would also have more serious ramifications for collapse due to the thermal effects over larger portions of the structure. These are the primary reasons for building code fire containment regulations, requiring fire wall and floor and door, test rated enclosures to contain the fire to one area for a number of hours.

. Prohibition of open access stairs, other openings in fire barriers, especially floors, and restrictions on the size of any area enclosed by firewalls, and thus exposed to a single developing fire would seem to be a must in high-rise buildings. Programs should be developed to properly seal any poke through openings made by trade people during renovations or installation of pipes, wiring or ducts.

The size of a fire is also a major factor that affects steel failure. "A large area fire in which flames involve much of the steel beam in a short period of time will heat the steel beam to its critical temperature more quickly. A so called "flash fire," suddenly involving a large area with flame can heat steel rapidly to its failure temperature." (Dunn, p.142). Because steel bar joist construction was used to provide this wide-open space within the Trade Center Towers, this additional hazard was introduced, compounding the collapse problem. These lightweight steel trusses are affected by fire sooner than heavy members and since they span such large distances, any failure becomes much more serious than a short span element. According to the preliminary findings of U.K. engineer Dr. A.S. Usmani et al., due to their length and slenderness the thermal expansion effects in long span, steel bar joists produce compression buckling in floors at lower temperatures than are presently compensated for in the fireproofing codes. "This sort of thing has not been considered in design of high rise structures with the possibility of multiple floor fires." (p.c.) Longer steel structural members expand a greater distance than short span elements and, as they are heated, slender elements can fail from buckling under compression, at temperatures that are still low enough (400 to 500 deg. C), that the steel retains most of its strength (Usmani); and lighter weight elements weaken in a shorter time due to more rapid increases in the steel's temperature to over 500 deg C. when heated in a fire.

Early Building Codes

A principle concept of the early, more effective, N Y City Building Codes, compiled in the 1938 version, which guided the construction of second generation, high–rise, 'fireproof' buildings is that fires would be, confined; boxed, within rooms or areas with fire walls and floors that had been tested against the possibility of collapse, the danger of passage of heat, flame, or large quantities of smoke and for 2, 3 or 4 hours. Buildings over 6 stories or 75 feet high were required to be 'fireproof', to contain fires for a number of hours without collapse. Thus restricted in growth, by fire walls, floors and doors and confined to one small area, and the buildings protected from collapse, fires could be safely fought from the inside and quickly brought under control, and the occupants could be removed to safety.

Theoretically and ideally in a fire-resistive, high-rise building, depending on its occupancy, enough passive fire protection (usually 'fireproofing' insulation on structural elements) would be utilized to withstand a complete burnout, of its contents before the fire or large amounts of smoke got out of any area of origin or any collapse would occur. A furnace test was developed to test representative samples of walls, floors and doors to determine their fire resistance rating in hours. Natural gas is used in these furnace tests to develop a, severe, ordinary combustible fire in accordance with a standard time temperature curve, which increased in temperature rapidly to over 1000 deg. F. and the increase gradually tapered off to over 2000 deg F. after 4 hours. Building component assemblies and their protection are rated in accordance with how much time (duration) they are able to withstand the severe heat of this Standard Fire Test, called ASTM E-119. Fire resistive buildings, depending on their occupancy class were, built according to the old code to withstand a complete burnout without collapse. For an excellent review of how practically every major improvement in the building regulations required a major fire disaster, read chapter 6 and 7 in Francis Brannigan's book Building Construction for the Fire Service 1971.

As an example of the collapse resistance of an early high-rise building we can refer to an article in the April 1948 issue of With New York Firemen (WNYF)

In the "First Skyscraper Fire" the Home Life Building at 256 Broadway, an office building of 16 stories built in 1893 was exposed to a fifth alarm, fire in an adjacent building in December 1898. With fire entering many floors simultaneously through the windows as the adjoining fire building collapsed, this early "fireproof" building became a "raging inferno from the tenth to the sixteenth floors". The primitive standpipe of the time was not effective and no water was able to be used to diminish the flames. "At the height of the fire, no one could predict whether or not the structure might buckle and hurl its tons of steel and masonry into Broadway". 'Despite the incessant rain, thousands of spectators remained all night in City Hall Park to witness the awesome sight, as the street was swept by fire'. It was not until 4hours later at 3 AM that the flames subsided. "It developed that the steelwork had withstood the ravages of the terrific heat in a satisfactory manner and buckling of steel members …had been confined to some few girders which required replacement. The building was restored and is, at the present time, still serving its original purposes." This fire led Mayor Van Wyck to plan a commission to draft a building code which led to the adoption of uniform standpipe and window protection regulations for high rise buildings, (WNYF April, 1948, p.5, 6)

Particulars such as materials used, thickness of fireproofing and fastenings, etc. for a particular type of floor or wall construction and the fire rating received by such wall or floor assembly in the furnace test, applied only to identical representative samples of the assembly tested. If the same thickness of fireproofing was to be used on another type of construction the whole new assembly had to be tested. Presently some builders refer to a certain thickness of sheetrock or spray on fireproofing as having a certain fire rating but this rating can only refer to the particular assembly that has been tested. For instance a ½ inch application of a certain spray-on fireproofing may impart a 4 hour rating to a large steel column but a much shorter fire endurance time to a lightweight steel joist. Sheetrock to keep its rating must be used with the same support system, fastenings and joint taping as tested in the furnace.

Unfortunately the standard furnace is only 14 feet by 17 feet and can not test long span construction and does not evaluate interactions between different building components and their connections during the acute stresses from the heat of a fire. Presently, "The combination of thermal load effects and structural load-carrying performance is ignored" and with the small size of the furnace and limitations on long span floor assembly testing "the performance under a specific load on a shorter member will not reflect the same loading on a longer member:" 'No testing facility exists that can provide full-or real-scale tests of combinations of building elements or their connections and interactions.' "(NIST, Analysis of Needs…, Dec 02, p.46)

That the long span, floor construction used in the Twin Towers, with up to 60 foot long bar joists, has apparently not been furnace tested for fire endurance under restraint is one of the possible major deficiencies in the WTC Towers. (NIST, May 03, progress report p.82) "The New York City Building Code does not prescribe how the required fire endurance rating is to be achieved. Rather the fire protection method is to be established by the Architect of Record and it depends, in part, on the structural materials used in the construction." (NIST, Progress…, p.70)

It's fairly common for architects in New York to be unlicensed according to E. Wyatt (NYTimes July16, 03). Minoru Yamasaki the architect for the WTC Towers was not licensed in N.Y.State. He employed the architecture firm of Emery Roth & Sons. Presently Architects, for registration in New York State are only required to take an Architect Registration Examination which is an oral test administered by the National Council of Architectural Boards which can take only about a half hour. Architects are certified for life after this conversational style test which supposedly covers the fire safety rules, design, strength of materials, construction and methods, site planning, building technology and building and fire codes. (Julia Levy, Sun July10, 03). I imagine each one of these areas would require a carefully constructed written test of about 4 hours to confidently ascertain competence. Just becoming a Fire Safety Engineer should require years of study and work in the area to gain the knowledge and experience to be qualified to design building systems. High-rise buildings architects and engineers should be qualified in all areas of construction and design before being allowed to practice on such large buildings.

Skilling Helle Christiansen Robertson (SHCR) the engineers that were contacted after a third alarm fire in1975 at the Towers to assess the structural damage and adequacy of existing flooring systems stated, "The only way to assure the existence of the fire safety of floor systems is to be found through the participation of a fire safety engineer and/or fire testing."(NIST, p.75) Since the floor systems were never subjected to the furnace test and in fact no furnace was large enough to accept these span long elements, all the fireproofing specifications arrived at were speculative and could not have assured the safety of the floor system. According to various Port Authority reports reviewed by NIST, the thickness of fireproofing necessary for the protection of the Towers bar joists had, at various times, been concluded or reported to or by the Port Authority to be ½", 1", 2", or 2-3/16" (p. 73, 74). Apparently, no one really knows what if any protection is effective in preventing the collapse of the long span, bar joist floor configuration used in the Towers. Large scale testing is needed to develop adequate knowledge and data for computer modeling of long span weaknesses in all types of construction.

Building Occupancy Classes which determine the fire protection required, depend on the amount of combustibles and life safety hazard expected in that particular type of occupancy. The collapse problem occurs when excess fire loads occur in occupancies not built to withstand an actual fire that could occur. Since the advent of computers, and their copious paper printouts office occupancies probably deserve to be upgraded as to their estimated fire loads and fire resistance required.

In the 1938 codes, columns, walls, floors, and doors, in addition to time-temperature, fire endurance tests, load tests and fire and smoke containment tests, were subjected, after the fire, to impact tests from a, fire hose. A solid stream, nozzle at 60 psi water pressure was directed at various spots on the sample being tested for ten minutes. This hose test was discontinued in the 1968 code revision.

Spray-on fireproofing materials- allowed in the present codes to protect structural steel- are easily dislodged and would, fail this early, hose stream impact, performance test. There is some question as to whether the present sheetrock enclosing stairways, corridors and, elevator shafts could pass these tests, but the fire service, at times, has to use high caliber streams which are more powerful than hand lines, in large area fire situations. These heavy water streams would certainly damage sheetrock enclosures and are sometimes even used specifically to breach gypsum board construction to control difficult fires.

I see no problem with performance codes as such. The difficulty is that, in the 1968 code, the fire endurance tests and other performance requirements were weakened. The dilemma in performance testing has been the prohibitive costs of doing full scale testing of new types of construction. As a result, full scale, as built, fire testing has, never been required in the building codes. The introduction of new stronger performance requirements, including as built, fire endurance and integrity tests for new types of long span floor and wall assemblies, and support columns -including their connections and structural interactions under fire conditions- and the re-introduction of the fire hose, solid stream or other impact test, or other impact or vibration tests to assure collapse resistance upon cooling and integrity of fire insulation on the steel, will go a long way towards making new high-rise buildings safe. Integrity of stair shafts and corridors should be assured by hardening such enclosures.

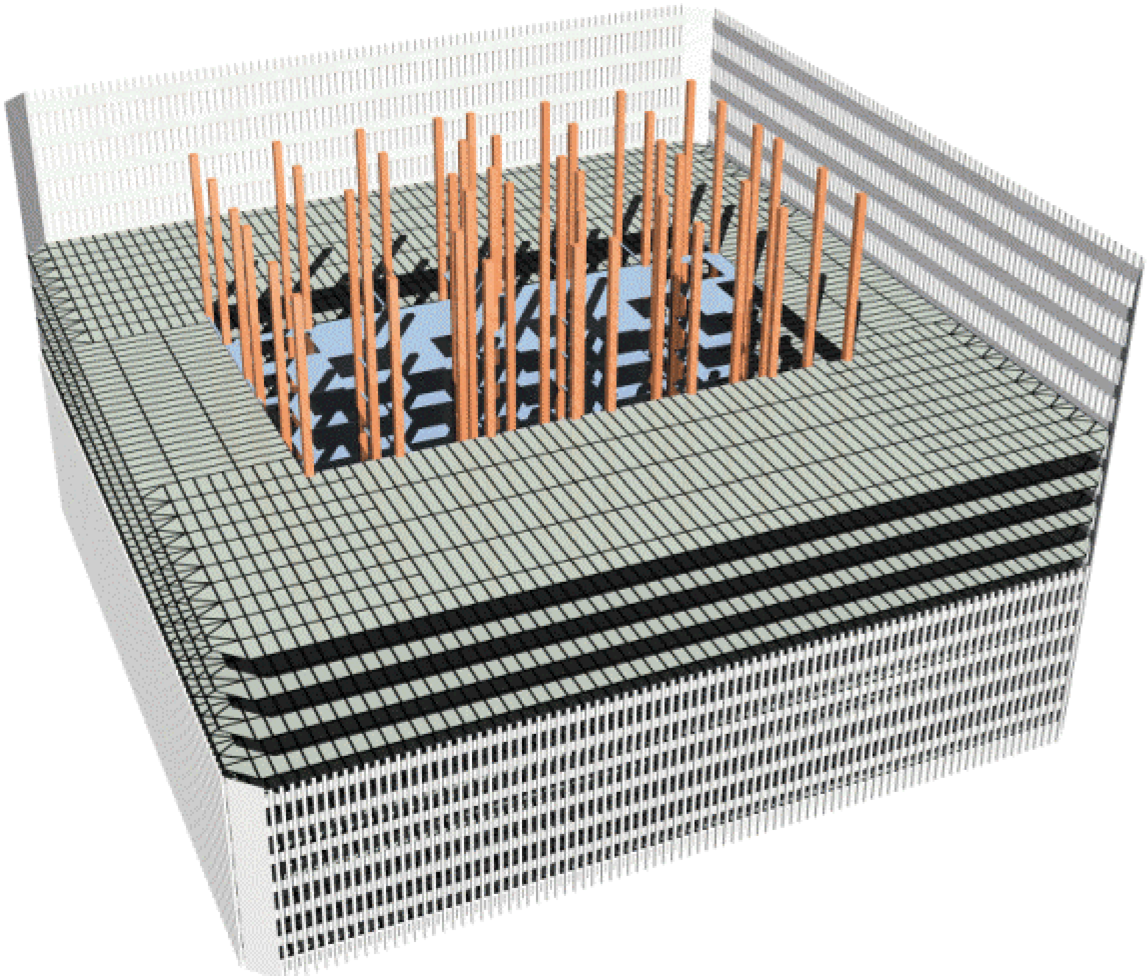
Presently large model experimentation is very expensive and the Federal, National Institute of Standards and Technology (NIST) with their extensive testing facilities and experience seems the most likely agency to undertake such examinations, if they can get adequate funding. It has become evident that the building industry will not, itself, support such expensive testing especially when it will limit their choices of design and construction. Indeed fairness, accuracy and dependability would seem to mandate that such testing be done by an independent, governmental agency.

All new types of construction should be subjected to this type of full or real scale, multiple floor representational fire testing.

Additional Reduced Performance Requirements, in the 1968 Code Revision and its Amendments

Besides attracting tenants, with large open areas and spectacular views; the competition to maximize profit, prompted owner/builders, to cut costs by increasing the efficiency of design by reducing building weight, bringing down materials costs, and reducing labor costs by speeding construction. Pressure from developers has led, over the years, to code changes, which: **One-** Allowed substitution of the inadequate, ineffectively applied, and easily dislodged, spray-on fireproofing, instead of effective, concrete, wire lath and plaster and masonry insulation, as previously required, for heat protection of structural steel. **Two -**

Allowed substitution of ‘sheetrock’, for the former concrete or masonry protective enclosure of stairwells and corridors. The gypsum board protecting the stairways and elevator shafts was destroyed by impacts at the WTC. **Three.** -Allowed concrete floors to be 4 inches thick rather than 6 to 8 inches of reinforced concrete necessary for effective fire containment. **Four**. -- Allowed unsprinklered floor areas between firewalls of up to 15,000 sq. ft. in high-rise office buildings—three times the area of fire that can be promptly and safely extinguished by Firefighters**. Five. --** Led to partial or non redundant ‘sprinkler systems’, which are sometimes, undependable, overextended and ineffective in large area fires. **Six.-** Permitted ‘controlled inspections’ which allow engineers or architects, hired by the owner, and subject to his removal, to sign off on inspections originally done by trained NYC Building Inspectors. **Seven**. -Allowed central air conditioning systems to service many floors with the possibility of spreading smoke to floors above and below the fire. **Eight.** – Led to <u>non</u> fire-stopped, concealed openings, in fire containment walls and floors, such as poke through openings for utility pipe or wire runs, and unenclosed access stairs between floors or removal of portions of floors, allowing fire and smoke to spread through the floors, and other areas above a fire. **Nine**. - Reduced fire protection ratings for walls, floors, columns and doors, reducing the time such assemblies could box in the fire and resist collapse. **Ten.** – Allowed “trade- offs” of reductions in fire resistance of fireproof insulation for the protection of steel in cases where sprinklers were installed. If you allow two different protections each of which is inadequate on its own, and if one of them fails they both fail. If sprinklers are down for repairs or otherwise become inoperable, a collapse and life safety hazard is created, by not having enough fireproofing to contain a fire or prevent collapse.



Construction WTC Towers

I found a description of the original plans for the WTC on the internet, and a brief review might be in order. It reads in part: “The 110 story twin towers will soar over New York City ... higher than any building ever constructed.” This 209 foot square building was to be a “bearing wall, space frame system” and an “innovation and major milestone in steel design”, a “new class of high-rise buildings”. “This wonder of the engineering world” used “maximum structural economy” to produce “increased rentable space made possible by long span construction which frees the areas of internal columns… this unique method of prefabrication and erection of the floor system will result in economy and maximum speed.” of construction.

These box column sections were to be two stories high - rather than three stories as built. The splice connections of the 3 ft 3 inch on center, perimeter wall, box columns were “to be field welded”. - They wound up being bolted . (I understand that columns are usually bolted during the initial erection than welded, later on, when the floors below the splices were in place and they could be easily reached.) In the WTC “Supplemental welds” were reportedly only used at the mechanical floors where ‘all the columns were spliced at one level’. (BPS)

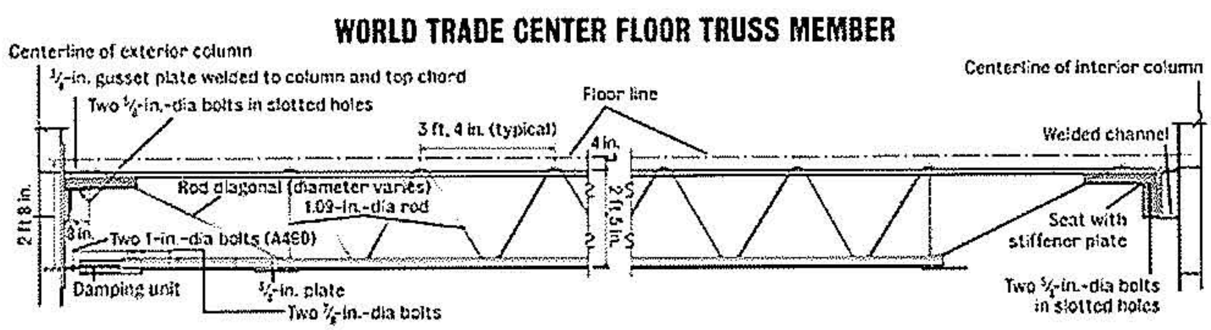
The twin tower floors were designed as a “one-way floor system”. The floor system was “reinforced by a two-way truss acting as a double layer grid, at the corners”; “This stiffens the corners by adequately tying the exterior wall sections together.”- The floors ware relied upon to tie the cross walls together, provide lateral support for and redistribute wind loads through the walls to the ground.

The angle steel, bar joist, trusses were to be 3 ft 3 inches on center, - instead of the doubled up bar joists with 6 ft 8 in. spacing which were installed.

“Sheer transfer is made by means of a single No. 4 continuous bar anchored at each panel point of the truss and centered on the 4 and 5/8 inch thick floor slab.” This bar was to be laid directly over each truss in one direction.- No concrete reinforcement is mentioned in the post fire, Building Performance Study, and the concrete floor was built 4 inches thick. “The entire floor deck could be pored in one operation without the expense of shores.” -The entire deck was about an acre, or 43,000 sq. ft. in size.

“The steel deck also serves as access runways for telephone and power cables.” According to the NIST progress report (May 03) “The 4 in concrete slab over metal deck had 3 in penetrating electrical header ducts in the slab… In 1966, ER&S wrote to the Port Authority that ‘with so many penetrations of the floor system the fire rating of the floor construction is of an indeterminate value unless tested. It is doubtful if it will meet a 3 hour test.’ “.

The Bar Joist Truss



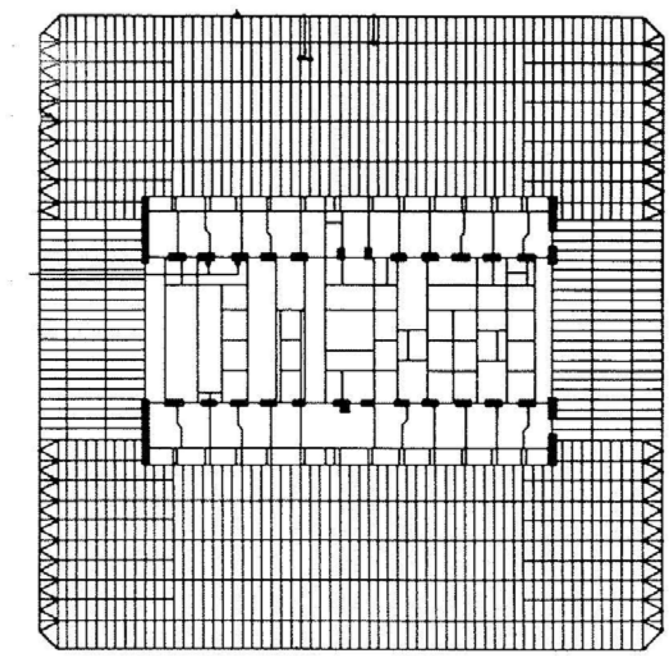
"Likewise the open web trusses allow complete freedom for horizontal runs of utilities between the floor and ceiling throughout 75% of the total floor system." -This open area truss void was also used as the return plenum for the air-conditioning system and would serve to spread fire and smoke throughout the fire floor and to floor above and below the fire. (Quotes taken from a report by Rapp, P.E. in Contemporary Steel Design)

As for concrete reinforcement and connections to the frames, there is no reinforcement indicated in the Building Performance Study (BPS) done by FEMA and the ASCE in May, '02. There were diagonal flat bars at the floor edges connecting alternate columns to the joists and they had some contact with the concrete, since they "were typically provided with shear studs, providing horizontal shear transfer between floor slab and exterior wall, as well as out-of-plane bracing for perimeter columns not directly supporting floor joists." (Civil Engineering, May '02) "Composite behavior" of the concrete with the trusses was attempted by extending the bent bars in the trusses above the angle iron, top chords so they would "act much like sheer studs".(BPS.2-3) This apparently has never been tested.

I see no evidence for mesh reinforcement in the slab. HERA engineer C. Clifton feels; "This would be very unusual as it would lead to more random and larger cracks in the slab which would not have been well appreciated by the client. On the other hand if this was not seen as a problem, then the leaving out of the anti-crack reinforcement would not have been of structural significance in normal operating or overload conditions. I don't know for sure that this slab mesh reinforcement was installed." (Clifton p.c.)

The Building Performance Study gives an excellent presentation of the main, as built, construction features of the WTC buildings and some collapse parameters. The Towers were built with a large, open area, outer ring of office space, typical of core constructed buildings. The Port Authority, however, used lightweight, open-web, bent bar, steel trusses (bar-joists), as floor supports, to span the distance between the closely spaced, steel box column, perimeter walls and the core area gravity columns which were interconnected by a network of steel beams. The core area in the center of the floor plan contained all the elevators, stairways, HVAC shafts, wire & pipe shafts and utility rooms.

Since the building had a square plan and the core area was rectangular, two opposite sections of the outer ring of office space were wider and had a clear span of 60 feet, while the other two sections had a span of 35 feet. Since the same sized steel was used in the bar joists the wider floor sections had longer joists which were inherently weaker and would expand and lengthen a greater distance if exposed to heat. “Steel will expand .06 percent to .07 percent in length for each 100 deg. F rise in temperature. Heated to 1,000 deg. F a steel member will expand 9 1/2 inches in 100 feet of length.” (Brannigan ’71, p.249) According to the BPS “an unrestrained, 20 meter [about 60 ft.] long steel member that experiences a temperature increase of 500 deg. C (1,022 deg. F) will expand approximately 110 mm [4 1/3 inches]. The bottom chord of the truss since it is longer will elongate a larger distance than the top chord at the same temperature. This will produce some ‘thermal bowing’ which will be enhanced if the lower chord is heated to a greater temperature than the top. This bowing will no doubt increase the tendency to buckle from compressive forces induced in the bar joists when elongating between the restraining columns.



Structural system for typical floor

If the floors deflected down enough and buckled from, restrained elongation and thermal bowing of the steel joists, or formed a plastic hinge by the top chord buckling or were weakening from the heat of a fire and sagged, or were impacted and buckled by a floor collapsing from above, they would produce about 3 times the horizontal, tensile forces acting on the wider 60 foot span sides than the narrow sides of the outside office ring. If a long span floor collapsed it would also impact the floor below with a stronger force than the shorter span sides, due to the increased weight of the floor.

According to structural engineer, C. Clifton; "The orientation of the floor on the East side of the South Tower was such that the long span trusses would have been the ones exposed to the combination of severe fire at the North East end of the East side and considerable impact damage to the South East end. This could well have destroyed any chance of two way action on that side, leading to one way action and the resulting catenary forces. If the East face did buckle inwards then that would be a strong indicator of catenary action playing a major role in the collapse." (Clifton, pc.)

Of course if two-way design was not built in, than there was no chance of its inhibiting the collapse. In a report from (SHCR) to the Port Authority after the 1975 WTC third alarm fire which spread- apparently through the floors- from the 9th to the 19th floor and did damage to the 11th floor where it affected 9000 ft sq. and "caused buckling of some of the top chord members, bridging bar joists and deck support angles." (NIST May '03, p.75), the WTC engineers stated, "In the *one-way* portion of the floor 'the concrete slab becomes the dominant element of the top chord.' Thus if the shear knuckle remains intact, [a big assumption considering the smooth nature of the ‘knuckles’ which are the tops of the bent bars projecting above the top chords of the trusses and could possibly easily disconnect from the heat weakened concrete]. “The structural integrity of the top chord is not required.” SCHR also reported “the structural steel top chord provides only a small increment in the diaphragm strength,” so the fireproofing may be omitted.” (p,77) As explained below, the buckling failure of the top chord is practically synonymous with failure of the load carrying capacity of a truss. If the knuckles detached from the concrete or the concrete failed in compression the floor would collapse if the top chords were buckled.

“The report also addressed the performance of the floor system in the 1975 fire, stating, ‘The fire of February, while reported in the press to have been very hot, did not damage a single primary, fireproofed element [this would not include the top chords which were not fireproofed]. Some top chord members (not needed for structural integrity), some bridging members (used to reduce floor tremor and the like), and some deck support angles (used only as construction devices) were buckled in the fire – all were unfireproofed steel.’ “(p.77)

Aircraft Collision - Fire Not Planned For

The twin towers were reportedly designed to take the impacts of large plane crashes. Since the effects of the fires after the plane crashes were, admittedly, not considered in the design of the building, one can assume the engineering calculations precluded the aircraft destroying enough columns to enter the building. According to a property risk assessment prepared for Silverstein properties: “The structural designers of the towers have publicly stated that in their opinion that either of the Towers could withstand such an impact from a large modern passenger aircraft. The ensuing fire would damage the ‘skin’ in this scenario, as the spilled fuel would fall to the Plaza level where it would have to be extinguished by the NYC Fire Department.”(NIST, Progress…, May ‘03 p.16). “Exactly how [the engineers] performed these calculations is apparently lost.” (Glanz and Lipton, Part Four). NIST continues to try to locate these documents relating to the Towers ability to withstand aircraft impact.

In fact the planes did destroy a number of columns and did enter the buildings, creating a cloud of atomized fuel mixed with air which immediately ignited inside. The resulting explosion blew out numerous windows around the perimeter of the buildings. It ignited most of the combustibles on the several floors involved, helped destroy the gypsum board, stairway and elevator enclosures, and dislodged ceilings and possibly floors.

This is still debated but, I believe only a small number of the interior core columns, if any, would have been severed by the planes, since these interior columns were stronger than the perimeter columns, and they were more rigidly restrained by the interconnecting steel beams and 5 inch concrete floors in the core. They were more widely spaced and the aircraft after passing through the outside wall columns would have been largely disintegrated before reaching them. Possibly, a direct hit by an engine could have severed one or possibly two core columns, but there is a big difference between an intact airplane hitting a steel column and a fragmented piece of a plane.

One engine, a landing gear and part of the fuselage passed through the South Tower and out the North perimeter wall with no apparent serious damage to the closely spaced, box columns. (BPS p. 2-31) Structural engineer Abolhassan Astaneh-Asi indicated, “The impact did nothing, pointing to a massive interior column from the South tower that he believes remained standing even after three-quarters of it was sliced away by a jet part.” (Jeffrey Gold, AP) This rectangular core area box column was 16 inches by 32 inches wide and composed of steel 1 and ½ inches thick, and was probably hit by a jet engine shaft, or a landing gear, the parts most capable of doing this kind of damage. Another core column Astaneh-Asi found that had been pierced by a jet engine was “like a bullet hole”.

The apparent visible damage to the South face of the North Tower would have existed, because in the fuel-air explosion, the aluminum cladding and window frames were blown out, along with the windows. The exterior fireball would have radiated heat back toward the building and began melting the aluminum cladding on the façade and discolored the wall, by soot residue. The only debris from the aircraft exiting the building from this side were life jackets and portions of seats and a landing gear which passed through the South façade but caused no visible column damage to the perimeter wall.

The atomized fuel and air explosion could also have caused a major portion of the gypsum wall board failure since it was powerful enough to blow out 3735 sq. ft. of windows on the West side and 4090 sq. ft. on the East side, of Tower 1. Both directions were at right angles to the plane’s trajectory. These windows were reportedly 8 times stronger than needed and were designed to withstand 140 mph hurricane force winds.

The sheetrock stair and elevator shaft enclosures and corridor walls could have been blown off the core columns with little damage to the columns themselves, just as the windows were blown out without damage to the perimeter columns. I think a large portion of the floor damage and certainly ceiling collapse over several floors also would have been caused by the fuel - air explosion which would have been directed in all directions, rather than the aircraft impact, which was directed only in one direction.

There is some evidence that the South tower twisted on impact from the plane and this may have been responsible for doors jamming in their frames and sections of ceiling being dislodged in areas remote from the impact area.

According to Prof. of Fire Protection Engineering James G. Quintiere and M. di Marzo and R. Becker writing in the Fire Safety Journal, Vol. 37, “It has been speculated that the impact knocked off the protective insulation [on the trusses]. We do not believe that this occurred to any extent in the fire region since a calculation of the bare steel chord [of the bar joist trusses] would suggest failure in 10-15 min. Again in the impact areas [which were not subject to fully developed fires] this may have happened, and testing needs to be done to assess the robustness of the insulation to impact. (FSJ, Vol. 37), my parenthesis [ ]

I have the impression that if as many core columns were severed – up to 40%, as surmised by some engineers, that the buildings would have immediately partially collapsed, since these core columns were strictly designed for gravity loads and did not have full moment connections to redistribute lateral, failure loads to intact columns and their network of beams, and the damaged sheetrock had little ability to transfer such loads. Such a large group of core columns failing would, let down support for large sections of core and outer ring flooring over many floors. I believe, an ”immediate drop of the core region relative to the perimeter frame, putting additional vertical load and tension force on the connections between the floor and supports to the core and perimeter frame columns" as HERA structural engineer Charles Clifton wrote, would have caused some immediate visible deflection (a drop of about '1 meter' according to Clifton p.c.) in the antenna, atop the roof, or at least some pull-in distortion in the perimeter wall as the core floors were pulled in and down, especially on the East and West short span sides, of Tower 1. There was no evidence of this.

These buildings however did not immediately collapse; it took almost an hour for Tower 2 and one hour and 42 min. for Tower 1, the North Tower, to collapse. According to Ronald Hamburger a structural engineer investigating the disaster, "We have reason to believe that, without the fire, the buildings could have stood indefinitely and been repaired." Professor Quintiere et al. of the University of Maryland thinks there is a direct correlation between the times of collapse and the documented thickness of the fire proofing, heat protection on the steel open web bar joists used to support the floors. (FSJ)

Smoke and Heat

Except for the smoke and heat caused by the unconfined, uncontrolled fire within the building, I doubt whether either building would have collapsed or whether such a large number of occupants and rescuers would have died. Even without total collapse, however, multitudes of people would have perished due to the destruction of the gypsum board enclosed stairway and elevator enclosures, preventing escape from above and facilitating the rapid spread of fire and smoke through the floor areas above the fires.

Some engineers surmise that the impetus for people jumping to their certain death, so very shortly after the impact, and from the very top floors, was a drop or sag in the floor and possibly other indications of impending collapse. This is apparently because they can see little visible smoke outside the building at the top floors, and do not believe smoke could spread up that high, that fast. At modern high-rise fires <u>any</u> visible smoke on the outside of the building is taken by the fire service, as an indication of severe conditions within. This is because of the tightly sealed windows keeping the smoke in. I haven't seen any of these videos yet but, the fact that smoke can be intolerable because it is so acrid and toxic, and excludes air and can radiate heat that's too hot to stand even though there is no flame, was almost certainly the cause of people being forced to jump. I don't think many of them thought for a minute that the building would collapse. Even experienced engineers did not think the Towers would collapse.

This smoke from the fire entering floor areas, above the impact zone, through gaps in elevator hoist way doors and defective stair shaft doors and possibly through cracks in the concrete floors or deficient fire stopping at 'poke through' or other openings, would have caused most of the life loss due to acrid toxic smoke and heat buildup on these floors, as indicated by the numbers of persons forced to jump from the windows. Judging from the number of windows in Tower 1 broken out, (more than 50 windows on 8 separate floors less than15 minutes after the start of the fires) by heat or by people attempting to obtain clean air to breathe (NIST p.38)

The top floors at any serious fire are very much exposed to rapid, severe smoke buildup, if there are any open vertical arteries or shafts extending upwards through the building. Super heated gasses, due to their buoyancy will immediately, rise rapidly up any open shafts, and, if they are not ventilated out at the roof, will spread out horizontally and begin filling the upper floors from the top down (called mushrooming).

Unprotected Openings in Fire Floors and Walls

Eliminating all the unprotected, non-firestopped openings in fire walls and fire barrier floors seem to be an unending problem in large buildings. One area or another are always under renovation and new poke-through openings for pipe or wiring runs, or ductwork are regularly made by contractors and building trades people, who may not understand the importance of the integrity of fire separation barriers, or how to re-install firestoping material. Fire and smoke can extend through a small opening in a rated floor or wall and in short order, spread a fire to another area or to the floor above. A report by the NY Board of Fire Underwriters after an investigation of the 1975 fire at the WTC which spread from the 9th to the 19th floors (report titled "One World Trade Center Fire, February 1975) stated:

"Unfortunately, no provision seems to be made for protection of openings in floors or walls. Consequently, some of the holes are not filled or others are filled with materials that disappear in the first seconds of a fire. It is ridiculous to spend time and money to prove that a floor or wall can withstand a two-hour fire and than allow holes to be cut in it that destroys the fire resistance". (Walsh, p.46) [This includes unenclosed access stairs.]

Present 'sprinkler' systems can be taxed and possibly overwhelmed if the fire extends through these openings in fire barriers to another area or areas, opening more spray heads than the system is designed for, thus reducing the available pressure throughout the system. Increased sprinkler system redundancy and capacity, is warranted, in high-rise buildings due to these seemingly ever present 'poke through' openings which, being concealed (usually by suspended ceilings) often go unattended. Unprotected 'access stairs', between floors, such as were apparently common at the WTC, are a severe hazard which allows rapid spread of fire and smoke to the floor or floors above the fire, smoke which, even if the building didn't collapse, could have been responsible for a loss of life.

The destruction of the Tower stairway and elevator enclosures by the airplanes at the towers would provide such an ideal channel for rapid, severe, superheated smoke spread to the upper floors. This smoke direction and buildup is one reason people are trained to go down the stairs and not up during fire drill evacuations. As a firefighter we learn this very early, especially at tenement fires, where it is axiomatic, that a man is assigned immediately to get to the roof, by way of adjoining building, aerial ladder or rear fire escape, to ventilate over the stairway to prevent and relieve heavy smoke buildup on the upper floors; this is a standard life saving tactic. At a serious fire this 'roof man', even if protected by a self contained air mask, would not be sent, up the stair inside the fire building to accomplish this task. He might not be able to make it or may become trapped by smoke and heat.

You don't need a fully developed fire over the whole floor area to quickly fill a room or area above the fire with toxic, acrid, non breathable possibly superheated smoke. If fire or smoke enters a stair, elevator or other shaft and there are openings from the shaft-ways into the floors, fire and smoke can spread simultaneously out of any shaft-way opening on every floor that has such an opening. Smoke and heat buildup will normally be most severe at the top floor first. If the stairway or elevator shaft has no ventilation openings above, it will fill with smoke and the pressure force the smoke out any available opening in the shaft, such as and defective fire door or the gaps around elevator shaft-way doors. With the damage to shafts at the WTC, if a stairway had open fire doors at any floor above the fire, especially the top floor, smoke would have rapidly flowed out there in large volumes.

As an example, at tenement fires we have a standard operating procedure, in case an apartment fire gets into a dumbwaiter shaft. A man would be sent to immediately open the bulkhead door and skylight on the roof, over the shaft to ventilate heat and smoke out before the fire and smoke could extend into the roof space, or out onto any floor with a defective dumbwaiter door. After the apartment fire was knocked down, the second Engine Company would automatically race up the stairs with a dry hose line to the highest floor possible before water filled the hose line. The line would than be taken into that apt. and a straight stream of water directed up the dumbwaiter shaft in an attempt to head off the fire before it took hold in the roof voids. That hose line would than be taken to the top floor or roof, because that was the most exposed area, for fire extension. All the apartments would have to be checked to see if fire had extended out any open or defective dumbwaiter doors into the apartments.

I am taking the suggestion of Ret. Firefighter and Fire Safety Director, Don Van Holt and recommending code changes for remotely controlled hatches opening over shaft-ways on the roofs of high-rise buildings. These ventilation openings could be activated from the fire command station on the ground floor. I am also recommending that all stairways and other vertical shafts be required to extent through the roof so that this ventilation can be achieved.

Weak Column Splices

The exterior (perimeter wall), steel, box columns apparently had week connections at the splices. The three story, column sections butted and were bolted to the columns above and below with four or six bolts, at each connection. It is evident from the failures that these bolted, column-splice-connections, were not of equivalent strength to the columns themselves; otherwise most of the deformations and failures would have occurred at random places, along the columns. According to the Building Performance Study "(BPS)…The propensity of [the] exterior columns to buckle would have been governed by the relatively weak bolted column splices…" (BPS, p. 2-25). Most of these column-splice-bolts were snapped by the aircraft impacts, tensile suspension forces induced in the floor joists, the loss of lateral support from the floors collapsing or later by the progressive collapse forces. A review of the pictures of the wreckage will reveal that the failure points occurred at these splices, rather than the columns themselves fracturing or being distorted. AISC specifications only call for 'sufficient rivets, bolts, or welding to hold all parts securely in place." (BPS, p.B-5).

Picture BPS, B-8-

A number of these exterior columns were knocked in by the impacts with the planes. If equivalent strength was designed into these connections the planes probably would not have broken through as many of them. It is, also not beyond possibility that, with the rush to complete the construction and the apparent lack of code compliance inspections, some of these bolts were left out or not secured properly. This could possibly, be determined by inspecting the bolt holes in the ends of the columns. Another question is; were the welds, reinforcing these connections, discontinued "near the building base" (BPS, p.2-3) by design or as construction expediency? These 'relatively weak' bolted-column-splices could have, in my estimation, played a role the initial failure of Tower two and possibly Tower one, by, allowing tensile pull-in forces created when the bar joists sagged to more easily buckle these columns inward.

Weak Concrete?

The 22 gauge, steel pan supported, concrete floors, supported by the bar joists, were built at 12-foot intervals vertically, so every perimeter wall column section- each being 36 feet long- was backed up by three of the 4-inch concrete, floors on edge. I can find no evidence of reinforcing having been used to strengthen the concrete. Steel reinforced concrete backup would have added significant rigidity to the columns under any impact from the exterior. If the perimeter wall, column-splice-connections and the concrete were more robust I doubt there would have been such easy penetration of the aircraft through the closely spaced exterior columns. Rather than "the plane slicing through the wall like a knife through butter', (according to one reporter), the plane would have been fragmented by the columns and floors. When the B-25 bomber hit the Empire State Building in 1945 the fire damaged several steel beams but the impact did not damage any steel columns. Granted it was a much smaller plane but, "One exterior column withstood the direct impact without visible effect." (BPS, p. A-10)

It is a distinct possibility that the concrete used in the floors was sub-par and did not provide enough back up reinforcement to the columns, a possibility which seems supported by the pulverized concrete evidenced in the debris pile. Adding too much water or air to the lightweight cement mix, during construction, makes for a more easily workable mix but a week friable concrete is the result. Stiff, properly mixed cement requires extra manpower and can exhaust the work crews, possibly slowing the construction progress. I can imagine the temptation to add extra water to the cement, to ease and speed up the spreading, floating and finishing work, over the one acre floor areas, especially since the original plans called for 'the entire floor to be pored in one operation without the expense of shores'. Pieces of concrete from the buildings should be tested for compliance with compression specifications.

During construction, I heard a rumor, which I dismissed at the time as being absurd, that they were using sheetrock instead of concrete in the floors in the upper levels of the buildings. However, if they had used two layers of 2 inch gypsum board over the steel pan floor and covered it with carpeting it probably would support the office loads. This substitution, or lack of reinforcing steel in the concrete itself, if it happened, would have compromised the structural strength, and possibly provided openings at the edges of the panels for fire and smoke spread through the floor. I never heard of a floor made of gypsum board passing any fire endurance test. According to structural engineer Charles Clifton (p.c.) "If the concrete was substandard over large regions this would have caused the effects [I mentioned above] as well as reducing the potential for slab panel action in the deforming floor. The result- 'of a substitution of concrete with gypsum board'--- would have been a more flexible floor in service with unacceptable deflection and vibration problems. If there were any levels in which this was reported to have been an issue then that would provide the best indication of substandard concrete." - I did read a report that, at a topping out party, the rhythmic dancing to the Mexican hat dance produced such alarming deflection in the floor on an upper area that a building engineer was consulted. (NY Times, The Height of Ambition, Part 5 Sept. 8, 2002). According to Clifton – 'floor vibration in a party on a lively office floor can be alarming however this could also be an indication of weak concrete' There were also numerous reports from the top floors of the building about swaying and rotating in high winds.

The Fire Affecting Collapse

The intensity of the fire (as it relates to building collapse), despite being ignited by jet fuel, was comparable to a heavy ordinary combustible material fire, after the explosion dissipated much of the jet fuel, and the remaining fuel burned off. According to the WTC Building Performance Study "…it is believed that almost all of the jet fuel that remained on the impact floors was consumed in the first few minutes of the fire." (p.2-22). According to Mr. Francis Brannigan author of Building Construction for the Fire Service; "The average person has no idea of the temperatures which can be reached in a quite ordinary fire. …temperatures in excess of 2000 degrees F. are the rule in severe fires."(Brannigan, 1971, p.245). So we can see that this fire was not out of the ordinary as far as temperatures developed are concerned.

The heat output and temperature of an interior fire is limited, besides by the combustibles available, by the amount of air reaching this fuel. The smoke and fire gas buildup could inhibit combustion by displacing air thus reducing heat transfer to structural steel. According to Prof. Quintiere et al.; "From the best available data as the fire diameter increases, the radiative fraction falls due to soot blockage." However, new test results by NIST may indicate otherwise. They used two different fuels in tests and report; "Despite significant differences in the sootiness of the flames from … two fuels, both types of fires produced similar temperature rises at the ceiling surface. However the bottom of the steel joist above the fire plume reached 600 deg. C twice as fast in the sootier fire, even though there was a larger temperature gradient along the joist length." (NIST, p.23) This temperature difference between the top and the bottom chords would have induced strong thermal bowing because of the differential expansion in the bar joists chords.

According to G. Charles Clifton HERA structural engineer, speaking of the fires in the Towers; "In my opinion, based on available evidence, there appears no indication that the fires were as severe as a fully developed multi-story fire in an initially undamaged building would typically be." And "…If the temperatures inside large regions of the building were in the order of 700+ deg C, [about 1300+ deg F.], then these regions would have been glowing red hot and there would have been …visible signs of flames." [Note; this is a temperature at which steel would weaken and fail.] However, he also states "It is likely that temperatures in some parts of the impact region would have exceeded 700 deg C for some or all of the time between impact and collapse, especially *on the South side of the North tower*," [my italics] and the *North east corner of the South tower.*

Since the stairway, elevator and other shafts were striped of their sheetrock enclosures, these open shafts would have served to draw fire, heat and smoke, up these shafts by the chimney effect. This flow direction may have limited flame spread out of the windows with consequent fire extension to the floors above the fire (outside extension) and protected the outside of the perimeter wall columns, from excessive temperature, while the radiant, heat from the fires, heated the inside surface of these exterior box columns. This may have produced enough differential heating to produce a thermal gradient in the columns and cause them to bow inward (thermal bowing) to a certain degree, but I doubt whether the perimeter columns could have failed, from heat alone, with out extensive flame projecting out the windows. C. Clifton, an expert in this field, agrees with me in this, and according to Dr. A.S. Usmani, Y.C. Chung and J.L. Torero of the School of Engineering at Edinburgh University, "The external columns had three of their faces open to the atmosphere that in the absence of external flaming is inconsistent with high temperatures. Therefore one must conclude that even if there were areas of high temperature inside the building, it is quite likely that the columns did not heat significantly (even if they had lost fire protection)." … "Considering these arguments …the scenario of a large number of columns suffering creep buckling almost simultaneously is not credible." (Usmani unpublished draft)

The immediate impact areas seemed *not* to have had a major severity of fire and this could have been because of the pulverized concrete and gypsum dust covering the combustibles and insulating them from radiated heat, and from most of the combustibles swept inward by the plane. The heat failure of the core columns is a major controversy since some theories postulate core column failure as a primary cause of collapse. However, I don't know the fire protection criteria used by the Port Authority but I believe the columns in this type of occupancy are required to be encased by insulating fire-proofing designed to protect them for at least 3 or 4 hours, or an hour more than floor assemblies. The amount of fireproofing possibly removed by the plane impacts is also unknown and needs further study.

It is possible, but unlikely, that the interior core columns could have been receiving more heat than the perimeter columns depending on how much fireproofing had been striped from them, and the intensity of radiant and convected heat they were exposed to. This could have been intensified from oxygen drawn up open shafts from below, combining with unburned gasses entering the core. But since there are no combustibles of any real consequence in stairways, elevator shafts, air shafts, bathrooms, or hallways, the core area would have had little fire load, except for the plane wreckage and any building contents pushed there by the force of the plane impact. The outer ring of office space would have had the major combustible loads, consisting of carpets, desks, chairs, computers, papers & books, file cabinets, partition walls etc. According to A.S. Usmani et al. "Internal damage of the core structure could have resulted in a significant increase in the oxidizer supply but numerical computations have shown that this would have also resulted in significant exterior flames."

Collapse risk

Consideration of progressive collapse in fire (proof) resistive buildings did not seem of critical importance, until now. "Prior to 9/11 there was no record of the fire-induced-collapse of such structures (fire protected steel-frame buildings) despite some very large uncontrolled fires." (BPS, p.8-4) NIST reports a total of 6 collapses worldwide in buildings over 21 stories with 3 of these occurring at the WTC complex. (NIST, Analysis of Needs…p.16). The good record in NY City is largely the result of the early robustly constructed, effectively fireproofed, steel frame buildings of the second generation, high-rise buildings, built to the 1938 code, practically all of which still exist. Lightweight core constructed buildings, built according to the relaxed 1968 and the later N.Y.City Building Code, might well be telling a different story, as evidenced by most of the serious high-rise fires occurring in these newer, core constructed, buildings. Evidence, the Twin Towers early collapse and the WTC building #7, which collapsed after seven hours due to fire alone. (see NIST, GCR 02 843 Dec.02 for list of fires leading to collapses)

In my estimation, in building #7, the lack of conviction of the necessity for effective fire resistant insulation on structural elements, led to the transfer trusses which supported the columns, having less fireproofing than the columns they supported. They were apparently following the code which requires more hours of protection for columns due to their importance in the structural stability of buildings and less protection for floor trusses. In situations where failure of floor support members (such as long span, bar joist floors) affects stability of columns, I feel these floor members should have protection equivalent to the columns. Prof. Usmani however feels that,

> "There is need to move away from blind 'protection' of structural elements from fire and base the fire resistance of structures on calculations performed against realistically possible worst case scenarios within a proper risk and reliability framework to enable quantifiably safer solutions to be achieved without excessive cost. This could mean applying fire protection much more judiciously, i.e. greater in areas of greater need and none where calculations show no need." (p.c.)

A problem with applying fireproofing only to critical beams or other elements is the possibility of misreading the blueprints and fireproofing the areas with no need and leaving exposed the critical areas. This could lead to problems with code enforcement if the inspecting agency is not aware of the reason for lack of fireproofing. . Why not fireproof all the steel and be on the safe side? Also the calculations may not be suitable for new types or configurations of construction unless long span, large or real scale testing is practiced.

The perimeter box column walls were robust and redundant in the plane of the walls, only supported 20% of their max load and I have the feeling (with no proof), could have remained intact even though several floors had collapsed and removed lateral support over a wide area, as indicated by having over 50% of the columns themselves severed on one face over several floors by the aircraft collision and the remainder still supporting the remaining floor loads. I surmise that the spandrel connections would materially strengthen the columns and delay buckling even though lateral support in one area were lost over one or more floors.

These exterior wall columns would not have failed, in that short a time, from the heat of this fire alone, and were weak in the direction out of the plane of the wall, just the direction most affected by the push-out and pull-in forces induced by thermal expansion and mechanical or heat buckling, and the induced pull-in (suspension) forces in the bar joist, floor supports. These perimeter columns could have contributed by thermal bowing in, increasing the compressive forces acting on the bar joist trusses than assisting the buckling forces in the columns. The core columns were more robust than the perimeter columns and had a higher fire protection rating than the bar joist floors and probably would have outlasted the fires.

There is some question about the core-column-connections and these may have been deficient, judging from the failure modes, appearing to have been cleanly broken off at the splice welds. These may have been vulnerable to lateral strain buckling and this may have been accentuated if all these butt weld connections existed at the same level.

Deficient 'Fireproofing' Insulation on Steel

The failure of a column, of course, is much more serious than a joist, since a column normally supports all those columns superimposed above it with their associated, girders and joists. It is because of their critical importance that columns are required to be protected by fireproof insulation for a longer time than floor beams. According to Zdenek P. Bazan, a fellow of American Society of Civil Engineers, "The analysis shows that if prolonged heating caused the majority of columns of a single floor to loose their load carrying capacity, the whole tower was doomed."(Simple Analysis, p1). Of course buckling failure of the columns by floor displacements could also cause this total collapse. These facts reinforce the importance of the fireproofing on the columns and joists, to prevent heat buckling or weakening; and the value of early fire control and therefore of containment of the fire by walls and floors, to limit the volume of fires to a size which can be extinguished promptly.

It has been shown that, at times, at the WTC towers, the fire resistance of both bar joists and columns was deficient, due to improper application and flaking off of sprayed on coverings in certain places. (NY Times, Science Sec. Dec 13, 2001). In the mid 1990s, "…Port Authority engineer Frank Lombardi discovered that the thickness of the fireproofing on the trusses would have been inadequate to protect the steel even if it had been applied perfectly. It was only half as thick as it should have been."(Glanz and Lipton, Part Five, Sept.8, 02).

"Despite the recommendations by Mr. Lombardi, thicker insulation had been applied to less than a third of the trusses in the twin towers by 11 September." (New Scientist.com 2/12/03) According to the Building Performance Study, at the time of the fire, the insulation on the trusses in the entire impact zone of Tower 1 were increased to 1 and ½ inches while only up to the 78th floor in the South tower was upgraded, and the impact zone should therefore have had only ¾ inches. (pp.2-12) Presently the NYC code requires 1 and ½ inches which according to Prof. Quintiere et al. doesn't even give a 2 hour rating. I don't know what criterion was used in the towers. "Our estimate of fire duration [time the fire would burn itself out] was about 2 to 3 hours (corrected). It has been implied that a three hour criterion should have been used for this floor assembly. Certainly, some factor of safety needs to be incorporated in relating fire duration time with test ratings or computed failure times." (Quintiere et al. FSJ) There is some question as to whether a 2 inch thickness could effectively be applied and maintained on trusses. The NYC Building dept has suspended use of bar joist trusses in high-rise buildings pending further analyses of the problem by NIST.

Sprayed on fireproofing which was allowed since the 1968 Code revision has been problematical in high rise construction. I now suspect that the hose impact test for fireproofing on steel was probably eliminated, in the 1968 code revision, specifically to allow the new spray-on fireproofing which than became widespread for steel insulation. Adhesion to the steel, however, with spray-on fireproofing is often ineffective, application uneven, and coverage sporadic due to obstructions. Portions of the friable fire protection are often dislodged by contractors working on other jobs. This defective protection often goes undetected since the areas where it is used are usually covered by ceilings, ducts or wall finishing materials. If deficiencies are found after the finishing work, it is very costly to repair, and every excuse for delay can be expected, since all the material covering it has to be removed or worked around to apply more fireproofing.

Imagine, fire companies, attempting to extinguish a large area fire and the hose streams knock off fragile insulation, on the beams and columns. This is no problem if the fire is small and brought under control rapidly, as most fires are, but if the fire grows beyond control and the hose positions have to withdrawn to protected, defensive areas because of high heat, an increased collapse problem will have been created by the fragile insulation, having been knocked off the steel.

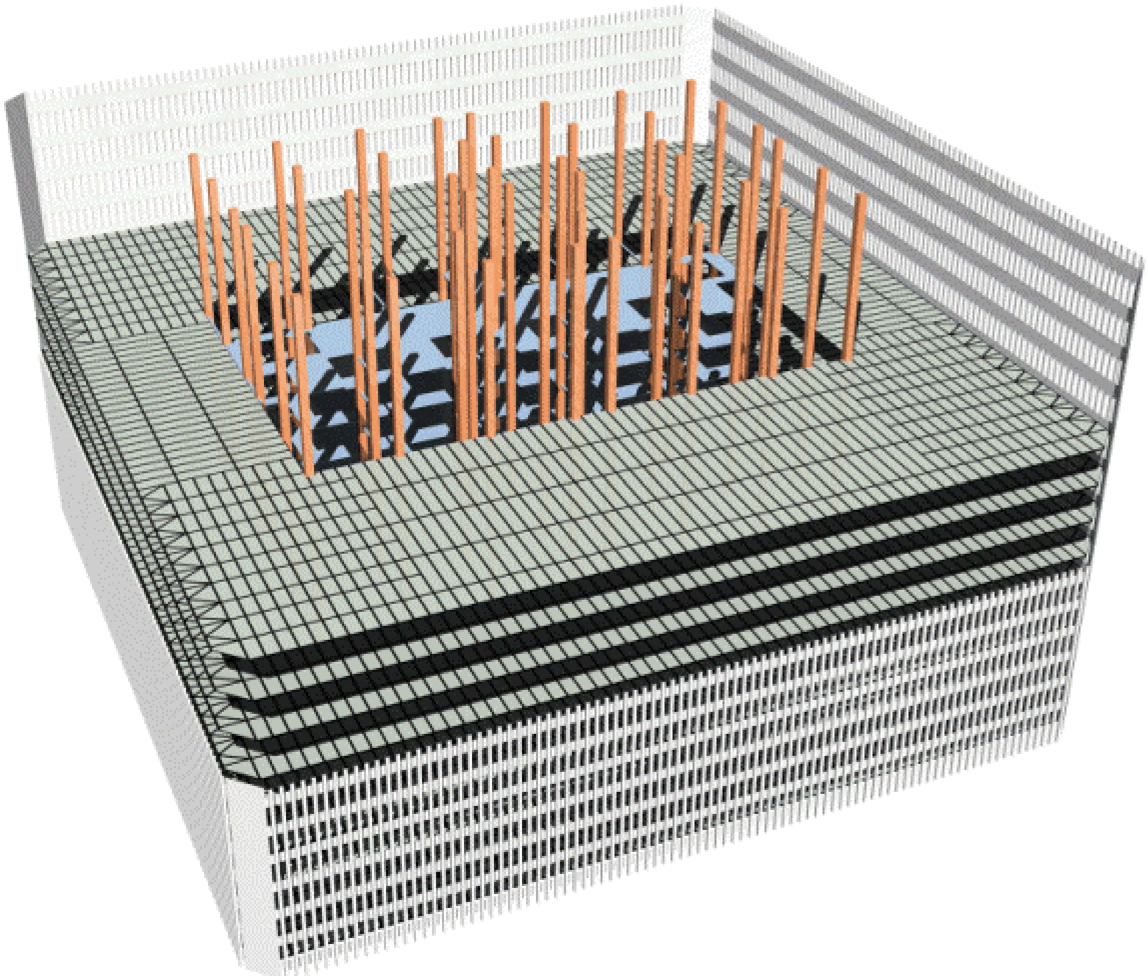
Long Span, Open Web, Steel Bar Joists

In the Twin Towers, light weight, long span, open web, steel bar joists were used, for the first time I believe, as the main floor supports in a high-rise building, instead of the conventional protected steel I beams, which are usually rigidly connected and supported at shorter intervals (19 to 30 ft) by columns, and spanned by reinforced concrete slab panels.

Instead of the columns failing first, due to heat, as proposed by some engineers, the weakest link in the WTC was these long-span, open web, steel bar joists. The position of these joists, over the fire and the small-diameter, steel elements of these joists would allow them to heat up to the steel failure temperature, approximately 630 deg. C to 770 deg. C.(about 1200 deg F.), more rapidly than the steel, box columns which would act as a heat sink and conduct some heat away. Another method of failure of the bar joists, not so much dependant on heat weakening of the steel, is buckling failure of the relatively slender bar joists due to restrained thermal expansion as described below.

It is axiomatic that a single column failure is usually more serious than a joist failure, but since bar joists are made of much lighter elements and every member in a truss is important for its structural strength, removal of a small portion of fireproofing from a truss would be more detrimental to the truss than a small piece removed from a column. Because of its greater mass a column would more readily conduct some heat away from a small area, and take longer to fail.

According to Deputy Chief, (Ret.) Vincent Dunn, FDNY writing in his book Collapse of Burning Buildings, “A large steel I-beam can absorb heat and take a relatively long[er] time to reach its failure temperature, while a lightweight steel beam, such as an open web bar joist, can be heated to its failure temperature much faster.” (Dunn, 1988, p142) “A [unprotected] lightweight steel bar joist truss may collapse after only five or ten minutes of exposure to fire.”(Dunn, video). This is hardly enough time for the Fire Dept. to get water on the fire, even in a low rise building. (Due to the number of buildings using exposed steel truss, roof and sometimes floor construction, I believe some means of plainly marking such buildings should be required for Fire Department safety, since such collapses can be sudden and without warning, when certain fire conditions exist)



The Tower's exterior, box column walls, besides being the bearing walls for the floors, were also shear-walls and, acting together with the floors, were the only lateral support for the building. They transmitted wind loads directly to the ground and through the floors, -which connected, supported and braced the walls laterally- to the cross walls. These exterior walls "together with the floors, formed a torsionally rigid framed tube fixed to the foundations." (Clifton p.3) Removal of the floor rigidity by the heat caused, buckling, sagging, spalling or cracking of the concrete, or softening or collapse of these bar-joists, over large areas would remove much of the buildings, lateral bracing, especially since there were no diagonals in the core, and the perimeter walls and core columns depended on the floors for lateral support, and lateral load redistribution between these exterior shear walls.

The lack of diagonal bracing in the core area (except in the hat truss in the top floors), is another apparent design deficiency, leading to numerous exit doors binding in the frames, after the aircraft impact deformed a large sections of the building frames.

Because of the weak joist-to-column connections inherent in lightweight bar joist construction, the flooring collapses could have progressed down through the building. This prospect is questioned as being improbable by Prof. Usmani because the development of sufficient kinetic energy would require "a large number of connection failures in a very short space of time" (p5) Some objections, to this point, are discussed below under the concept of 'rip failure'.

I characterize these connections as inherently weak because, since these chords are relatively lightweight, to make the joist-to-column connections stronger than the truss chords themselves would just move the failure points from the connections to the chords.

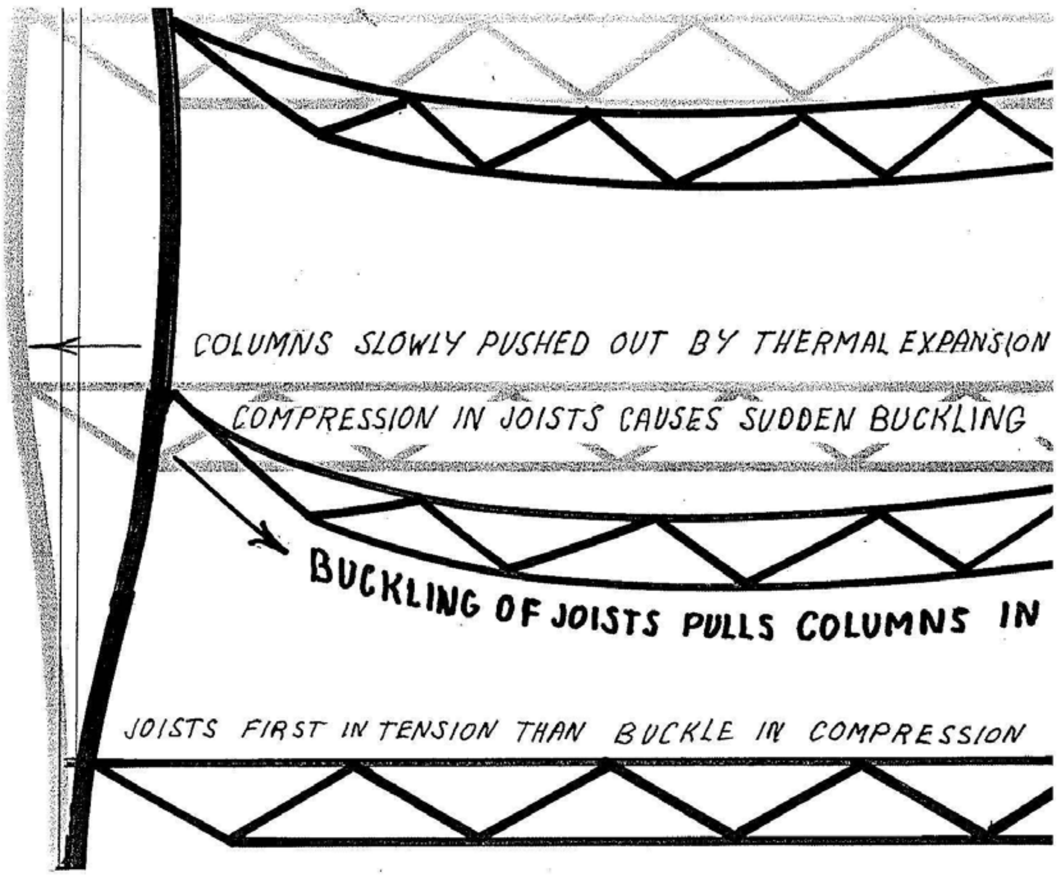
More serious than removing the columns lateral restraint against buckling, sagging in the bar joists, by the steel softening or failure of the truss rods, by compressive forces causing buckling of the top chord inducing a plastic hinge in the joists, could generate horizontal pull-in forces. As the floors sag the truss chords could act in suspension, weighed down by the floor loads, pulling in on the supporting walls and columns. Large or real scale tests are needed to determine the actual magnitude of these forces, and determine the mechanism of collapse, in the Towers in order to develop possible corrective modifications.

Initial Collapse Mechanism - A Design Deficiency

A new theory has been proposed by British engineers A.S. Usmani, Y.C.Chung and J.L. Torero of the University off Edinburgh School of Engineering, UK. They postulate thermally caused expansion and resulting compression buckling in the steel bar joist floors, as an origin of sudden removal of lateral support for a number columns on more than one story causing buckling failure of the perimeter columns. They used 'non-linear, two dimensional computer program which is able to model the real structure of the Towers with great accuracy'. Their analysis posed the hypothetical question "had there been no structural damage [from the aircraft] would the structure have survived fires of similar magnitude?" Their analysis found that, without postulating any defects in concrete or steel, and due to "geometric thermal expansion effects" and "material effects of loss of strength and stiffness" in the slender, steel bar joist trusses that "the structural system adopted for the Twin-Towers was unusually vulnerable to a major fire". This design problem would be the initial, fire-induced collapse mechanism operating on the Towers structure, since thermal expansion of steel members begins immediately, as the steel is heated, and when the steel still has 90% of its design strength.

Professor Usmani gives the following description of the sequence of mechanisms responsible for the collapse: (p.c.)

Before the fire, the loads on the floors are resisted by bending, (The top chord and composite concrete floor are in compression- maximum at midspan; the bottom chord is in tension, and the diagonals are in tension or compression depending upon orientation.



1. During the fire the whole composite floor is in compression (because of restrained thermal expansion against the columns) and the external columns are pushed out by expanding floor.
2. With continued heating, compression builds up to the critical in-plane buckling and the floor buckles.
3. The column looses lateral support at the buckled floor and pushes the floor in.
4. As the column pushes the buckled floor in, the floor experiences very large deflections (so if the column was pushed out 6 inches by the floors, the floors will have deflected by 2 feet when the column returns to its original position – and this is nothing to do with the loads , its simply to accommodate the expanded floor – the formula is in my paper "Fundamental principles of structural behavior under thermal effects", Fire Safety Journal 36, (2001) p.721-744) The floor is still able to carry the loads on it by a combination of bending (compression in the composite concrete and top chord - remember that buckling has relieved the compression from thermal expansion – and tension in the bottom chord) and tensile membrane or catenary action. With increasing deflections, the contribution of the former [bending] diminishes and that of the latter [tensile membrane or catenary action] increases. The tensile forces from catenary action are anchored to the columns and therefore contribute to pulling them in further and may also cause connection failure (as many have assumed, but more detailed analyses and tests are needed to investigate this further)
5. This sequence occurring in a number of locations could precipitate collapse (this is the speculative part as only "one" planar frame was analyzed). A 3 dimensional analyses of the structure (including the "top hat" or "outrigger" story height trusses at the top

> floors) is required to understand the exact collapse mode resulting from the mechanisms discussed above. A local floor buckling failure will probably still not cause collapse as the structure could redistribute the load away from a collapsing column by using columns in tension “hanging” from the “top hat” truss. Therefore a number of such buckling events would be required before this redistribution capability of the structure is exhausted and the collapse begins. This also ensures the collapse to be reasonably uniform, i.e. beginning at many places simultaneously (as it seems to have been – in the North tower at least). (Usmani p.c.)

Prof. Usmani et.al. found that compression buckling due to the increasing steel temperature, elongating the slender, long span, bar joists, between the rigid columns, occurred at low temperatures (400 to 600 deg. C) even before significant weakening of the steel. The “small amount of heating will make it [the bar joist] buckle out of plane because of restrained thermal expansion”.

> “This is not to do with buckling found in other tests (such as Cardigngton, where the lower flange of the I-beams actually buckled (yielded in fact!) at temperatures of 100-200 deg. C. However the floor slab was stiff enough to the end. But Cardington had smaller spans (stiffer) and no multiple floor fires and very low loads.” (Usmani, p.c.)

The Professor found that for the configuration used in the Towers, and for extensive fires of more than one floor, the various forces induced in the joists and columns by heating effects were sufficient to buckle the columns by removing lateral support simultaneously over several floors. He however contests that induced pull-in tensions by the floor buckling had much to do with the initial buckling of the columns.

> “It is more a case of columns ‘pushing in’ than the floors pulling them in, as the columns are only able to remain vertical by ‘leaning’ against the floor diaphragm. Remove the floors and the columns collapse! This is the …fact of elastic stability (Euler’s law) …” (Usmani P.C.)

Long columns of the same diameter are inherently weaker than short columns. If you double the unsupported length of a column by removing one floor which is providing lateral support, you reduce its compressive strength against buckling to about 25% of what it was originally, depending on whether it is completely elastic or turns plastic. (This buckling effect on slender columns can be demonstrated by taking a 2 inch straw and trying to buckle it with your finger, and comparing it with a 12 inch straw. The longer straw will more easily buckle and collapse.)

Commenting on this Prof. Usmani observes:

> “Remember that the Euler buckling load reduces to ¼ by the loss of one floor and to 1/9 with the loss of two! The catenary effect is by contrast opposite, the greater the deflection the lower the horizontal force on the column.” (p.c.)

Dr. Usmani believes that the stage where the joists soften enough to induce substantial suspension induced, pull-in forces is <u>not</u> attained because the columns buckle and collapse before the steel temperature, and consequent softening, required for such strong catenary action (acting as a cable in suspension) is reached. The pull-in effect, however, is difficult to determine since the floor stiffness may decreases due to compression bending, thermal buckling or heat softening effects, and the pull-in forces may increase even though the deflection is also increasing. This effect can be seen in a comparison between Usmani's graphs Fig.11 and 15 where as the floors continue deflecting between 1300 and 1700 seconds the pull-in tension forces are increasing. However the catenary relationship - described below by Francis Brannigan (increasing deflection with reducing pull-in)- is apparently only effective when the truss looses all stiffness and acts like a cable in suspension. Something is evidently happening, before heat softening, to reduce the stiffness in the floors as they bend in compression or due to thermal bowing or heat weakening effects or a combination of these mechanisms or possibly some other reason exists for loss of stiffness as the deflections increase. Perhaps long span composite construction behaves differently when heated due to steel stretch out leaving the concrete unable to provide compressive resistance. More tests are also needed to explain this increasing pull-in effect which apparently has not been seen in previous testing.

I believe the reduction in horizontal catenary effect found on greater deflection when the steel is soft enough to act as a cable is at least partially due to the angle of floor attachment narrowing as the floor sags. This horizontal force may also increase due to the building contents sliding down the floor and redistributing the loads as the slope increases. Also I believe that if the top chord of the bar joist buckles at one spot along with concrete failure that a plastic hinge will be created which may eliminate any stiffness in the floor which could attenuate the pull-in forces.

Prof. Usmani points out his results are based on the extensive Cardington experiments which "have shown that connections remain under compression while the fires burn …as a result of restrained thermal expansion'. They [the connections] do snap in tension, but this occurs on cooling, when the fires have burnt out, which clearly was not the case for these buildings." (I wonder whether this 'snapping' also applies to cooling with water. I was under the impression that steel freezes in place when cooled with water and this method was used to prevent the hazard of collapse.) The Cardington tests were not long span structures and used steel 'I' beams, composite with reinforced concrete slabs which are more robust than the slender, lightweight bar joist trusses which I surmise would more easily weaken and buckle, and form a completely plastic hinge.

According to Prof. Usmani, the columns buckle inward, not principally from the catenary forces but primarily from the sudden removal of the lateral support for the columns as the bar joists buckle from the compression forces.

> "There are tensile membrane forces in the floor which may contribute to the inward movement but are not entirely responsible for it." (p.19) " As the [bar joist floor] membrane stiffness reduces, [through buckling ?] the outward movement of the column is arrested and the stored strain energy in the column makes it recoil with an increasing rate of inward

> displacement pushing the softened floor system back in….The column eventually over-shoots the original position significantly…" (p.30)
>
> "For many of the cases studied … collapse is initiated before the steel truss reaches 500 deg. C . . . … collapse occurs at truss temperatures of 500 deg. C. for all three-floor fire scenarios and at 700 deg. C. for two-floor scenarios…" (p.29) "In none of the analyses carried out failure occurred by outward buckling of the columns. (p.30)
>
> "This suggests that even if the fire protection to the steel trusses had survived the impact, the failure temperatures required could have been attained."(P.29).

Dr. Usmani et.al., emphasize the hypothesis of "rapid loss of lateral restraint to the column" (p.8) and the 'rebound effect' as main destabilizing forces, with tensile membrane action contributing, but not the main force.

> "As explained above the catenary pull-in is a much smaller effect than the loss of lateral support. In general I would conjecture (without proof) that if a column is strong enough to survive the loss of lateral support it will survive the catenary pull-in and also in this case connections could be designed to snap before the tensile forces build to any significant level." (Usmani p.c.)

On this point I have to differ with Dr. Usmani's explanation. The graph Figure # 15 clearly shows that, after those floors which are being heated, expand and buckle in compression, the induced compressive forces in the floors relax, change into tension (pull-in), and increase in magnitude until runaway, column collapse begins at about 1700 seconds. This occurs in the two floor, fire situation as the floors are heated by the fires. The 'fire floor' itself remains relatively cool even though the fire is directly on that floor. This is because of the concrete's insulating qualities protecting the bar joists. The two floors above the fires are being heated and first begin to expand and be subjected to thermal bowing and to increasing compression against the stiff columns.

As Dr. Usmani persuasively points out, as the floor is first heating up;

> "In case of an expanding floor or beam, the beam is getting compressed because it itself is pushing against stiff restraint, when buckling occurs in this situation some of the compression is released (as the compression comes from the beam or floor getting longer and trying to fit into a smaller space, the buckle allows it to bend out relatively gently, and now the longer length is absorbed in the deflection, thus relieving some of the compression). The release in

> compression allows the member to continue to carry out of plane loading as a beam in bending." (Usmani p.c.)

This release of the increasing compression, upon the floor bowing downward, can be seen in Dr. Usmani's graph (Figure 15) as the compressive forces decrease after 1000 sec. until they reach 0 at about 1300 sec. As these compressive forces are reduced the column which has been pushed out by about 15 mm naturally returns with increased velocity (Fig. 14) to its original position. The reason the column eventually goes past this position to 50 mm and eventual buckling is under contention. Using his graphs I contend that the loss of stiffness upon bending a certain distance induces tensile pull-in forces, even before the truss softens enough to act like a flexible cable. The Prof. believes the 'use of the numbers from his analysis to make any estimates of velocity is not correct as this was a static analysis and it didn't cover dynamic inertial forces'. He feels it would take a 'dynamic analysis which included inertia and damping forces' to determine the relative effects of tensile pull-in vs. rebound and lateral restraint removal forces.

From his graph in Fig 15, the pull-in forces begin to increase, at this time (about 1300 sec.), as the floors, switch from compression into tension, exactly the time at which the column after returning to its normal position begins to deform inward from the vertical (from Figure 12). We have to assume the columns are being pulled in by the indicated increasing tension in the buckling floors, and not "rebounding" inward, of their own momentum. The column only moves back about 12 mm, in about 50 seconds (from Fig. 12). That is hardly enough velocity to gain any appreciable momentum, and from my experience after a structural steel element deforms it does not even return to its original shape much less 'overshoot' its original position, unless it is quickly released. There may be some eccentricities eventually increasing buckling forces due to column lengthening but this would not come into effect until the column was pulled past the neutral position by lateral forces.

Dr. Usmani in explaining the 'rebound effect' believes that;

> "Truer figures for velocity and momentum etc. can be obtained by doing a dynamic analysis which includes inertia and damping forces. When I talk about rebound it is essentially pointing to the fact that the column is acting like spring when paused by the floor and if the floor buckles, the strain (potential) stored in the column will be converted to kinetic energy and WILL rebound. It is difficult to know if this would be a significant effect or not without a dynamic analysis." (Usmani p.c.)

The columns certainly can not be leaning on the floors which are pulling in on them. The columns are leaning on and compressing the cool floor (the 'fire floor' on which the fire is burning). At about 1500 sec. the compressive resistance of the cool floor comes into effect and decelerates the column's inward deformation. (Fig. 14) The increasing compression on this relatively cool floor can be assumed to release the runaway collapse at 1700 sec. as this floor buckles from the increasing tensile forces pulling in the columns.

One can assume these tensile forces in the floors had much, if not everything, to do with the initiation of the bending of the column after the release of compressive forces and column's lateral support was effectively removed. Granted some of the buckling force on the column will come from the weight of the perimeter wall and attached floors above compressing the elongated column and this effect will greatly increase after the column bends out of plane a significant distance. The relative proportion of forces, through time, which affect the columns from compression to lateral pull-in and to the compressive loads from above, should be examined further.

According to the graph (Fig.15), the increasing tensile force from the deflecting floors, pulling in the column, when it reaches about 100,000 (N) in each of the two heated floors, assists in buckling this restraining cool floor, starting the runaway column collapse. The compressive force, acting on the cool floor, by the two hot floors pulling-in the column, and just before the cool floor buckles is about 170,000 (N). As seen in the graphs these pull-in forces naturally decrease, as the column buckles and deforms as this releases and reduces the tension in the floors. This tensile, pull-in forces, however, remain positive for some time even as the column collapses.

This graph only shows how large the forces, - just sufficient to buckle the floors or columns - became. Exactly how great these lateral pull-in forces could become, with stiffer restraining floors or columns, is undetermined, especially given the many different modes of bar joist failure. We could also conjecture - without any proof - that if a bar joist floor develops a localized buckle by say failure of the concrete and top chord at one spot, that it can act as a completely plastic hinge at that location and the only forces acting on the columns would act in tension in a direction corresponding to angle at which the joist attached to the column, since there would be no floor stiffness left to direct these forces otherwise. These tensile forces would naturally 'reduce practically to 0' once the columns collapse and release the tension. (p.19)

Picture- Figure # 21 Usmani's report

The graph in Figure # 21 of Prof. Usmani's report shows the compressive and tensile forces generated in a different situation where the columns didn't fail and release these forces. This graph shows that, in the 'floor above' the fire, the tensile forces progressively increased and went off the chart above 100,000 (N) and did not return for the 3,500 seconds covered in the graph. From graph Fig. 22 we see that the column lateral velocity inwards clearly *follows* the floors downward deflection velocity. I consider these lateral forces could have been decisive to the early column destabilization which started the progressive collapse. The increase in the number of floors deflecting would increase the column pull in effect exponentially. Since the perimeter columns were strongly interconnected by the spandrels which would redistribute compressive and lateral loads on the columns and such columns carried only 20% of their designed loads, I doubt, they would fail so early in, response solely to Euler's law, based only on removal of lateral support.

I agree that, since the floors were providing lateral support to the exterior columns and core columns and in effect were integral to the stability of the whole structure, and that even without pull-in action, removal of lateral support for a long enough stretch of these columns, by enough floors deflecting or detaching, could allow the weight of the building above to buckle both outer perimeter and possibly the interior core columns, letting down the entire upper portion of the building. This widespread column failure would inevitably trigger further sequential failure and total progressive collapse, from the "dynamic overloads from the falling superstructure as a rigid body" (Usmani p. 4)

However, the number of perimeter columns and area relieved of this lateral, floor support, to cause such a collapse, without the help of the horizontal pull-in forces, would be considerable, because of their redundancy and because of the spandrel girders connecting these columns. “The utilization factor for gravity loading is reported to be very low 20% for exterior columns and a moderate 60% for core columns.” (Usmani p. 4) This strength was demonstrated by the fact that more than half of the 59 columns themselves were removed over several floors, in each perimeter wall, impacted by the planes, without further collapse. The columns above the destroyed sections acted in tension still supporting the floors, by the rigid vierendeel truss action, diverting the forces around the damaged area, and no further buckling ensued.

Column buckling would more readily be caused by the horizontal component of the suspension forces developed, as the bar joist floors bow downward, loose their rigidity, buckle and possibly form plastic hinges, and /or sag, rather than solely by failure of these floors removing lateral support. If the sagging floors remain attached, they add vertical and horizontal loads to the columns, loads which could not affect the columns if the floors were detached from the freestanding columns.

With a 2 dimensional model we really should be talking about single joist sections instead of floors. As Dr. Usmani suggests “To achieve a firmer conclusion a 3D analysis would be necessary” … and this “is the next logical step to take this investigation further.” (p.30)

The Three Floor Fire with Double Columns

The following explanation is for the three floor fire scenario in accordance with Usamini’s graphs Figures 24, 25, 27, & 28. To coincide with F.D. N.Y. nomenclature I am changing the floor descriptions from Usmani’s report. The ‘fire floor’ is the lowest floor on which the fire is burning (usually the floor of origin). The ‘floor above’ the fire floor is the 1st floor above the ‘fire floor’. The ‘2nd floor above’ is the 2nd floor above the fire floor. The 3rd floor above is the 3rd floor above the fire floor etc.

In order to follow this report you should have Prof. Usmani’s graphs. I found that if you make copies of the graphs on transparent paper you can overlay them where appropriate and greatly simplify comparisons.

See diagram below.

With the increasing compression in the hot floors (the ‘floor above the fire’, and the 2nd ‘floor above’ which are directly over fires and being heated from both upper and lower side), they expand against the columns and at about 300 to 400 seconds, they start to deflect downward and the compressive forces begin to decline.

The top fire floor or ‘3rd floor above’ the fires is an exception being heated only from the bottom, it is initially in tension (Fig.27) due to not expanding as fast as the columns are being pushed out by the 2 lower floors (which are both being heated from above and below). This 3rd ‘floor above’, -still expanding as the 2 hotter floors buckle and retract,- is subjected to additional compression as the column is released from push-out by the 2 failing hot floors and it begins to return to its original position. At about 900 seconds the ‘third floor above’ also suddenly buckles, rapidly releasing its compression against the columns. This buckling adds another floor which is reducing compressive forces rapidly; this allows acceleration of the column’s velocity inward, as it continues recovery back towards its original position.

Column displacements reduce to zero at about 1000 seconds at the same time the hot floors compressions have reduced to zero (this suggests that it is the compressive lateral force removal which has allowed the column to return to its original position).

At about this time (1000 sec.) the 3 failing floors change over to rapidly increasing, pull-in tensions and the columns begin rapid inward displacements. This velocity is reduced as the bottom cool 'fire floor' resists the pull in and rapidly increases the restraining compression forces slowing the column's inward displacements.

At 1600 seconds this cool lower 'fire floor' buckles in compression relieving (reducing) some of the tension in the hot floors. (less so in the top, hot '3rd floor above' which is furthest from the cool buckling 'fire floor'). As the cool 'fire floor' rapidly buckles, it also switches into tension making 4 floors in tension pulling in the columns. This immediately again increases inward column velocity which levels out (1700 sec.) and than begins to reduce. This inward column movement slows partly because of the resistance offered by the 'top non fire floor' (the '4th floor above') coming under compression from the column deflection, and allowing the tensions in the hot floors to begin to increase again.

At 2250 sec. the columns, especially at the 'top hot floor' (3rd) rapidly increase the velocity of column inward displacements (Fig 28). As expected the floor tensions rapidly reduce especially in the top heated (3rd) floor (Fig 27). This sudden velocity increase probably coincides with the 'top non fire floor' buckling (not on graphs) in compression, reducing resistance to inward column displacement again, and adding to pull-in forces from the 3 hot fire floors which had leveled out and the cool 'fire floor'. That's 5 floors which have buckled inward adding to the pull-in forces on columns. (Fig 25, 27, 28)

At about 2800 seconds, as the columns are increasing lateral displacements inward, the columns buckle (reach elastic limit) suddenly and begin 'runaway' collapse with increasing velocity inward and again relieving lateral tension in the floors. This floor tension however remains positive until after 3500 seconds as total column collapse proceeds. The pull-in forces are positive until final collapse.

Prof. Usmani's study found that a 500 deg C. fire could cause collapse in a multiple floor fire, based on his analysis of the geometric effects of thermal expansion causing bar joists to buckle in compression, as the joists expanded between the columns.

Dr. Usmani:

> I would say that greater insulation may not have been of much help in the case of these trusses as they would have reached the buckling temperature sooner or later, as you very rightly indicate they will heat up quicker because of lower mass. A better option for WTC would have been an alternate lateral support system for the columns in fire (such story height trusses at appropriate intervals, or a diagonal bracing system in the plane of the floors). (p.c.)

To my thinking and due to inherent complications the only way to prove the mechanisms and magnitudes of these forces and deflections in novel steel frame buildings and whether they can be compensated for is by realistic fire tests on similar long span building assemblies.

Since Dr. Usmani has shown that long span bar joists buckle at lower temperatures, due to the mechanical effects of expansion rather than the weakening effects of higher temperatures, it seems that any current insulating capacity may be inadequate to prevent collapses in long span bar joists. Since we haven't allowed floor collapse, under fire conditions, as an option in high-rise construction for over 100 years, it seems we would have to prohibit long span bar joist construction for floors.

It has been commented that it may be possible to relieve the pull-in forces before column failure by designing the floor-to-column connections to snap upon floor buckling. If you if you use this method to prevent column failure, does it not practically assure the progressive collapse of the long span floors? It seems we had a similar argument over 100 years ago with the idea of the 'fire cut' on wood floor joists to release the joists, before a collapsing floor could put leveraged torque on the brick bearing wall and buckle or break it, or pull the wall in by tensile forces. In the early 1900s, with the increasing building heights and the persistence of progressive floor collapses, -possibly aided by such self releasing mechanisms, - fire induced floor collapse became unacceptable. This serious collapse problem was solved in these non fireproof, higher buildings by inventing fireproof floors and walls that could withstand the effects of fires.

Other Possible Mechanisms of Bar Joist Collapse

Other mechanisms of collapse have been put forward; some due to deficient strength in the concrete or the loss of integrity of the bond between the concrete slab and the bar joists, some due to deficient fireproofing, or different truss elements at different locations failing.

Prof. Quintiere et al. did an analysis of the temperatures developed in the Towers and the effects on the bar joist trusses used. He correlates the collapse times with the different thickness of fireproofing on the trusses in Tower 1 and Tower 2 and 'Finds the results extremely telling and believes the results show the root cause initiating the collapses.' "Our hypothesis is that the truss rods are the weak link because they have the lowest cross sectional mass and the fire would increase its temperature the fastest."… "It appears the insulation thickness on the truss rods was deficient and caused the heating of the steel that led to weakening and collapse."

Professor Quintiere et al. postulate that the long thin, transverse truss rods would fail first from compression buckling as the rods were heated and 'restrained elongation' occurred buckling the rods as the fires progressed; subsequently the "whole transverse [bridging]-truss failed, sagging downward". "Sagging of the floor deck followed, developing some membrane action in the steel deck, and possibly disconnecting the concrete within the floor span…"… "Due to the ductility and continuous nature of the deck itself and the continuous nature of the transverse-truss bottom chord, the floor probably…..stayed hanging from the main-truss bottom chord." 'The main-trusses were simultaneously being challenged in a similar way by the fire. Restrained elongation of the top chord superimposed a bending moment on the truss as a whole increasing its tendency to deflect downward with possible <u>buckling of the top chord in the vertical plane</u>. With most of the compressed diagonal rods already buckled the "beam" or "truss" behavior no longer existed and the <u>top chord acted as a cable</u>, from which all the other members were hanging downwards.' [my underlining]

Professor Quintiere et al. postulated “increased pressure on the inner edge of the end seats while increasing the tension in the bolts, actually bending and shearing the bolts due to enlarged rotations.” ‘Total collapse of the floor ensued, placing load on the floor below.’ “With several floors simultaneously involved in fire, this would cause several floors to load the floor below. He assumes the connections at the columns could not tolerate the load and progressive collapse ensued in the floors. (Quintiere et al. Fire Safety Journal Vol.37, p 707) This progressive collapse mechanism is disputed by Dr. Usmani et al. who indicate that connections would not fail until cooling caused contraction in the joists. This conflict needs further analysis, especially in light of the photographic evidence (NIST p.39) of disconnection in the East wall of the South Tower at floor 83, and the apparent multiple floor failure in Tower 1 nine minutes before the collapse.

Due to the fact that a small number of perimeter columns failing would not lead to progressive collapse we have to explain successive failure of the bar joists. Although I agree with Dr. Usmani’s theory, except that we differ on the magnitude and effect of the pull-in forces from the deflecting trusses. These induced tensile forces in the floor joists along with other forces could also have disconnected some joists or portions of floors from walls which fell onto the floor below. Buckling of the floor impacted, could have easily added to the lateral tensile forces pulling the perimeter walls in by buckling the concrete and top chord of the lower trusses. Prof. Usmani disagrees:

> "Although the floors do go into tension after the buckling of the floor trusses, the very large deflected shape the floor adopts at this stage is a very efficient load carrying shape for tension and the tensile forces are unlikely to be high enough to cause connections to snap and there are so many of these with considerable redundancy to fail in large enough numbers." (Usamni, p. com.)

This leads us to the possibility of successive failure of the bar joist trusses.

Sequential Failure

It is possible by all the joists being heated a similar amount that all the joists fail at once over a whole floor but a more likely scenario is sequential failure of the bar joists. A bar joist is a truss, and the failure of a section of bar joists can lead to successive failure of adjacent joists due to load transfer. "In fact, successive failure of trusses appears to be the rule rather than the exception."(Brannigan, p.46) The fact that the bar joists were connected by transverse bridging trusses and intermediate, deck support angles (BPS p.2-4), could aggravate the successive failure prospect, since these bridging trusses and support angles along with the steel pan and concrete could transfer weight sequentially to adjacent bar joists as each failed.

As stated by Mr. Frances Brannigan in his book Building Construction for the Fire Service, "Not only can such trusses heat up rapidly due to their high surface to mass ratios, but by the very nature of their design and installation they are susceptible to collapse over a wide area. Every connection is vital to the stability of the truss. …the inherent instability of long thin trusses makes it necessary to tie the trusses together. By means of these ties, torsional loads generated as one truss fails can cause structural failure of trusses not affected by the heat" (1971 p. 277).

An interesting sequential cause of bar joist failure in the Towers could be due to thermal expansion of the slender trusses reaching their compression failure point and buckling in succession. This would be similar to column buckling by 'rebound' effects as reported by Dr. Usmani et. al.:

> "Note the floor truss system is very long and quite slender and not really designed for in plane forces (membrane or axial compression). Initially the floor system may expand (for multiple floor fires thermal expansion will dominate over bowing as the floor is heated from top and bottom) resulting in pushing out of the external columns. Eventually the increasing membrane compressions from restrained thermal expansion and the high slenderness of the floor will lead to buckling of the floor and further increase in deflections. The significant change in the geometric shape of the floor system will lead to

> further reduction in axial capacity, leading to a rapid loss of lateral restraint to the column."

If a long section of bar joist floor trusses is expanding and pushing out the columns, this loss of push-out compression, in one small section as the bar joists buckle -and the switch to pull-in tension in these floors - would transfer this tensile force to adjacent columns through the spandrels. This effect could increase the compression forces in the adjacent expanding joists starting a sequential buckling failure along the floor. This sequential buckling would be assisted by the weight of the buckling joists and their loads being simultaneously transferred to the adjacent expanding bar joists through the numerous interconnections

Picture pg 39 NIST Progress report May 03

This change from compression to tension could also lead to a section of joists detaching from the columns, possibly indicated by the apparent sag in the 83rd floor of the East face of the South Tower, as pictured on page 39 in NIST's progress report. I would call such a connection detachment which is expanding or traveling a 'rip detachment'. I imagine such a rip detachment could travel along one entire side since it would transfer an increasing floor weight sequentially from joist connection to joist connection. As each connection snaps it would immediately transfer the floor load to the adjacent connection. If a floor failure started at one end the weight of the detached floor could, as it sagged down thus act sequentially and with some gaining momentum and speed as it impacted each joist-to-column connection, sequentially continuing the 'rip failure'.

This section of bar joists disconnecting from the columns would transfer their weight to the adjacent joists by way of the various interconnections between these buckling bar joists, at the same time these adjacent joists are having increased compression from the recoiling columns thus greatly increasing chances of sequential 'hinge' buckling assisting the 'rip failure'. Tensile forces in the joists could also assist the 'rip failure'

Besides the sudden tension being induced, in a section of expanding floor, by joist 'hinge' buckling, such detachment could also have resulted from a cool section of the floor being put in tension by the adjacent floor expanding and pushing out the columns.

The question arises as to whether such a one side 'rip' floor failure would cause collapse in the floor below. In writing of the possibility of connection failures setting off a chain of progressive floor collapses Prof. Usmani says "This theory is improbable as it relies upon a large number of connection failures in a very short space of time to set off floor collapses with sufficient kinetic energy." (p.5) However, a 'rip detachment' may not have enough kinetic energy to propagate floor failure in the floor below but the furnishings sliding down the tilting floor could place a large impact overload in a concentrated area on the floor below especially since any accumulated sprinkler water would also be dumped onto the floor below. I would think such a sequential 'lean too' collapse could lead to a progressive floor collapse, either by causing buckling of the top chord thus inducing pull-in forces in the floor below and destabilizing the columns or by sequential impact floor failure, inducing either a pancake, 'lean too' collapse or a progressive 'rip' failure in these lower floors.

Column Buckling by Catenary Forces Induced in the Joists

Catenary action- strength of the suspension forces ?

The forces not under dispute are the pull –in due to loss of strength in the steel causing the bar joists to effectively sag into cables in suspension. This collapse mechanism, - the pull-in of the columns by catenary forces, therefore must be examined, as they come into effect in the last stages as the steel looses all stiffness. If the joist-to-column connections held, as the bar joists buckled forming local plastic 'hinge' buckles or sagged due to loss of strength, the truss chords would act in suspension, inducing pull-in horizontal forces on the columns. According to Mr. Francis Brannigan writing in Building Construction for the Fire Service; "Steel members which sag due to fire will try to carry their loads as suspension members. This causes large horizontal [catenary, tensile] forces: if these are transmitted to the fire wall, it can be destroyed."(1971, p.274)

The internal, forces acting within a bar joist truss and the one-way catenary forces created when the truss sags into a suspension member from the heat of a fire, can be conceptualized if you visualize a hammock tied between two trees. I have had several trees bend over, and one (rotten one) collapse when I placed my weight on a hammock. The more tightly you string the hammock the more force will be created, pulling in the trees, as you place your weight on it.

Now, if, before you placed your weight on the hammock, you wedged a long broom handle between the trees, above the hammock, to prevent the trees from bowing together, you would essentially have the truss situation. The broom stick would be the top chord of the truss (in compression) counteracting the forces (from the hammock -the bottom chord-, in tension), pulling the trees together. Holding the broom handle with both arms to prevent it from bending, as you lay in the hammock, would mimic the truss rods between the top chord in compression and the bottom chord in tension.

Breaking the broom handle or letting it bend, and buckle or otherwise loose its strength in compression would allow the horizontal suspension forces from the hammock ropes and possibly the broom handle itself to pull the trees together. The broom handle (the top chord) could, after it buckled, also act as a cable in suspension after the truss deflected downward, pulling the trees (columns) together. This could happen from the top compression chord or the truss rods buckling from heat caused by restrained elongation. Also having the hammock stretch, as would happen if, say it suddenly turned to rubber, would cause the persons arms to pull, and buckle, the broom handle, thus pulling the trees together. (This would mimic the bottom chord of the steel truss elongating or stretching due to being overheated by a fire.) This is catenary action where a member acts as a suspension tie, like the cables on a suspension bridge, and the force acting in the horizontal direction is the catenary force. The concrete also acts in compression and would assist stability unless it cracked, crushed, or detached from the top chord.

Catenary forces would also be induced if the trusses twisted out or flopped over as the bridging trusses and diagonal braces interconnecting the joists failed. 'Flop over' can be visualized if you imagine trying to break a piece of wood lath or Venetian blind slat by placing the narrow side against your knee. The slat would be hard to break until it ‘flopped over’ and the wide side was against your knee, when the slat would immediately break.

As described by Mr. F. Brannigan,

“The bottom chord of a truss is under tension. Recall that a tension member is like a rope. One break precipitates failure. Conversely, the top chord of a truss is under compression, therefore it responds exactly like a column.”(’92, p.528) The struts or rods support the top chord (the column) and effectively reduce its length. Removal of one rod doubles that column section’s length and reduces its load carrying capacity to 25% to 50 % of what it was. So buckling or weakening of one rod can affect the whole truss, and we can see that buckling of the top chord a truss or heat weakening turns the whole truss into a suspension system which puts horizontal tensile forces on the connections to the wall. The perimeter columns could, after bowing in, break at the weak column-splice-connection bolts due to the buckled or sagging trusses pulling the columns into the building.

The Formula for Catenary Force

The formula for determining the horizontal force created by catenary action, in one truss, as given by Mr. Brannigan (1971, p274) is:

$$H = \frac{wL^2}{8S}$$

where H is the horizontal force in pounds, w is the deadweight in pounds of a strip of floor 1 foot wide and as long as the spacing between trusses, L is the truss span in feet, and S is the sag in feet which can be assumed to be between .07x L and .09 x L. As can be seen in this formula, the horizontal, pull-in force increases as the square of the truss length (span) and inversely proportional to the sag distance.

Therefore if the span is doubled the horizontal catenary force is quadrupled. So, long span bar joists exert much greater horizontal forces if they sag or buckle into a catenary, than shorter span elements. Also, as a truss fails, the initial small sag exerts greater horizontal forces than the subsequent increased sag, due to their direct inverse relationship. It’s possible that an intact truss floor buckled into a catenary by restrained expansion or by a floor falling from above, will, for the reason it has no, heat caused sag, produce higher tensile forces acting on the wall, than if it did have some sag. Dr. Usmani’s compression buckling due to restrained elongation would initially have very little sag. This should be explored further, since a sound floor buckling or a floor collapsing onto a sound floor may apparently induce even stronger pull-in forces than if the steel just lost strength due to being weakened by heat, after expanding.

Each floor weighed about 3,200,000 lbs and divided by the floor area of 207x207 ft or 42,849 sq.ft. = 74.7 lbs. per sq. ft. self load.

If we take 100 lbs as the sum of the live plus self load per sq foot at the WTC; 60 feet as the truss length; sag in feet as .08x 60 and, 6 foot 8 inches as the distance between trusses than:

w = 6 ft 8 inches x 100 lbs, L = 60 ft, S =.08 ft x L

w = 666.6 L x L = 3600 S = .08 x 60ft = 4.8

$$H = \frac{wL^2}{8S}$$

Horizontal (catenary force per truss) = 3600 x 666.6 divided by (8 x 4.8)
2,400,000 divided by 38.4 = 62,500 lbs.
Horizontal force per truss = 62,500 pounds

Since there are 30 trusses, the force on the wall from the whole floor acting as a catenary after the trusses sequentially failed could equal

30 x 62,500 lbs = 1,875,000 lbs

With two floors sagging, the force acting on the East wall could be over 3.75 million pounds, but since the bolts and welds connecting the joists to the columns would fail at a force (to be determined), the approximate buckling force that could be developed on the wall, before the joist-to column connections failed, remains to be established. This would be an exercise for engineers; to ascertain whether the joist-to-column connections would fail before the wall would be pulled in, - in the case of one floor, two floors and three or more floors becoming catenaries- also along what length and on how many floors the columns would have to be subject to these pull-in forces to cause column buckling of an entire wall. Also on how many floors and over what area the columns would have to become unsupported, without catenary action to cause buckling. The more serious condition would seem to be where the floors remained attached, further loading the perimeter wall both laterally and vertically. Dr. Usmani's work which utilized ' a intact model of the Towers with no structural damage from a plane or weak concrete, indicates that if more than one floor is affected by fire, the rapid loss of lateral restraint to the columns over several floors would lead to collapse. He however disagrees that the catenary force is a main contributory force in the column failure at the Towers,

> At low deflections a sound floor is not a cable! It does not use tensile membrane mechanism to support the load, it used bending resistance. That's where the WL(sq.) /8 comes from (it is the maximum moment from a uniformly distributed load at the midpoint of the truss). If however the truss has no bending resistance left then the moment (still WL(sq.) /8) is resisted by the horizontal reaction (H) at the end of the floor (at the connection) and the same force H at the midspan of the truss (acting in the opposite direction) times the distance between them (which is the deflection S). This formula is only

> applicable to cables. The floor system even in its buckled shape is not a cable! And retains significant bending capacity, so your calculations are misleading. Also in my analysis the floor goes into tension after buckling and therefore can NEVER be the primary cause of column collapse, it can only add to the effect, the exact value of this needs more work to determine." (Usmani ,p.c.)

Actually in Prof. Usmani's graphs, (Fig 12 & 15) which only analyze the static forces and do not cover the dynamic inertial forces, the floor only goes into tension, as the column recovers to around its original position about 1300 seconds. After the floor buckling the floor rapidly reduces in compressive force allowing a rapid return of the column to its original position; the floor immediately switches into increasing tensile forces as it deflects downward pulling in the column in well before the column buckles. If there are any other forces that would deform the column inward past its original position, I believe they would be from heating over a gradient causing thermal bowing, where thermal expansion of the inside face of the column from fire effects, bows the column inward.

Also the facts appear to be that the floors did detach in Tower 1 and at least started to detach in Tower 2 as indicated in the NIST pictures of the 82nd and 83rd floors. This would indicate tensile forces greater than Prof. Usmani's analysis indicates did develop. This may have been because of the many different manners of buckling possible. True catenary action would occur if the steel in the truss softened from the heat and in effect became like a cable or rope in suspension. Dr. Usmani's buckling formula accounts for some stiffness remaining in the floor and thus some of the loads being transferred vertically downward through the column and some of the bending resistance counteracting the pull-in forces.

> "The reaching of the membrane buckling limit [from the compressive forces caused by thermal expansion against the columns] simply implies that no further in-plane loads can be applied. However the geometry changes are accelerated by the buckling and the floor which had been in compression begins to sag and eventually sag so much (with associated material degradations) that the structure now begins to find it easier to support loading in tension. This is a gradual process and does not suggest a loss of all bending capacity until much later. You can imagine that even in a sagged state, the top floor 4 inch concrete composite with the truss top chord continues to take compression coming from loads- the thermal expansion related compression is decreasing- and the bottom chord of the truss continues to take some tension to provide bending resistance, in addition to the increasing tension from the tensile membrane effects. So the formula you are using is not applicable. It is for a cable with zero bending capacity, The non linear analysis shows this, as you rightly point out that my numbers are lower for the pull-in

> tension than yours (because the analysis automatically accounts for the bending capacity).” (Usmani p.c.).

Still the increasing tensile forces evident in the graphs must have a cause. Other mechanisms of buckling is possible. When I think of buckling I think of complete loss of bending strength as when a stick snaps as you bend it over your knee. This type of buckling may be important to further understand brick and wood joist construction floor collapses. The stick is still attached together and so still has tensile strength but has no stiffness laterally. This type of buckling where a plastic hinge has developed at one point of the truss could have happened in the tower bar joists. This could have been because of defective concrete, disintegrating in compression, lack of reinforcing causing cracking and separation of the concrete or ineffective bonding between the concrete and the top truss chord, which had just the tops of the bent over bars projecting into the concrete. If the concrete cracked, separated from the joists or disintegrated in compression, the only compression resistance left would be from the top chord of the truss. Something new may be operating in long span construction. For instance the heat caused expansion and steel stretching as tensile forces increase may be expanding the steel faster than the concrete. This could reduce the compressive component of tensile membrane action increasing the pull-in forces or it could break the composite bond between the steel and the concrete with a similar effect.

This top chord could buckle at a stress point and once this happened if the concrete also failed, the joist would have a plastic hinge at this location. The joist would have no bending resistance against deflections and could only impart pull-in forces acting directly in line with the joist sections where they were attached to the columns. This force would have a horizontal component whose magnitude depended on the location of the hinge, and the angle that the joist was attached. If the buckle (hinge) was near the column most of the force would be downward and the pull-in would be reduced after deflections of the column inward changed this angle to nearly vertical.

I don’t know if these forces would be the same magnitude as if the joist became a completely flexible catenary but they certainly would be greater than if some rigidity (bending resistance) remained throughout the joist. Both of our formulas may be inappropriate for the actual situation in the Towers. A new formula to cover the local plastic hinge scenario may be needed.

Progressive Collapse

The full scale long span or representative tests and computer modeling, which we hope can be done by NIST, will, determine the actual causes and mechanisms of failure in the progressive collapse of the Towers. This determination is necessary to determine corrective actions to prevent similar fire-induced collapses in the future.

I surmise that the steel bar joist floors deformed out of plane due to thermal bowing due to differential expansion in the top and bottom chords, and restrained expansion or forming flexible hinge buckling, and/or sagging into catenary action, they went from compression to suspension. This failure mode, possibly assisted thermal degradation of strength and stiffness, or impact loads, put pull-in forces on the perimeter wall columns and the core columns. This force caused the exterior columns and possibly some core columns to buckle inward and fail at the column-splice-connections.

According to the, BPS report (p.B-14), "Tensile catenary action of this type of floor framing members and their connections has <u>not</u> been a design requirement or consideration for most buildings". MIT engineers postulated that the joist connections to the columns failed first (ie. "The trusses sagged ….pulling the bolts inward….the bolts popped off as the trusses fell") [NY-1, Oct. 27, 02]. Actually according to BPAT engineer W.G.Corley (11/04/02) 'the connection of the exterior columns to the floor trusses was welded, directly to the exterior columns through gusset plates at the top chord of the trusses. The bolts in the top and bottom chords were used for erection purposes and remained in the final assembly.' There were also diagonal bars extending from the alternate columns (between those supporting the trusses) back to the top chords of the trusses, and they were anchored into the concrete.

- Picture BPS, p B-10-

According to Clifton p.c. "It is likely that loss of restraint to the perimeter columns on the East face of the South Tower… occurred over several levels, so a loss of 75% of the pre-impact compression in these columns is likely." With a utilization rate of just 20% this still would not buckle a stand alone column and the perimeter columns were interconnected by the spandrels and could be supported in tension from above.

Possibly as the long span, bar joists buckled they opened the steel pan floor and let down some of their concrete loads and live loads onto the floors below. This may have been the reason for reports of 'fire moving downward' 12 minutes before the collapse of Tower 1.

A pancake, U- shaped collapse or a lean-to, 'rip' collapse of a long span, bar joist floor could impart a concentrated impact load on the floor below. Since top chords of trusses are in compression and, just as in a column, bending of the top chord can cause buckling, and truss failure, an impact load could buckle the top, steel chord any where along its length to collapse a truss or series of trusses into suspension which can, either shear off the joist-to-column connections or induce tensile forces directly pulling in the walls, and initiate bearing wall failure by buckling. Twinning or doubling up the bar joists increased the shear strength but would allow double the, pull-in forces to be developed, before joist-to-column connection failure, while not reducing the time to joist weakening buy heat buildup. Doubling the joists could have also interfered with application of the spray-on fireproofing.

It is doubtful whether the lightweight, long span bar joists on the floor below could sustain the impact, of a floor or its concrete falling from above, without buckling into suspension or shearing off the column connections. Either one or both of these could lead to progressive collapse of the building. Steel I beams and properly reinforced concrete floors have more resistance against buckling since the spans are usually shorter and the top compression flange of the I beam is continuously rigidly reinforced by the solid web. Connections to columns can be strong and rigidly restrained because the beams themselves are robust, and the composite floor panels with inherent tensile strength, can benefit from two way action. Clifton gives an excellent description of the difference between one way and two way action, in reinforced concrete floor construction.

> There is a fundamental difference between the behavior of a joist type floor system compositely bound into a concrete slab under one way and two way action. For example, one of the slab panel tests reported in [5] was on a Speedfloor panel, which is the NZ version of the bar joist floor system. Under two way action this loaded test panel survived 180 minutes [3hrs] of Standard Fire Test conditions [the New Zealand tests are effectively the same as ours]. It would have gone longer but the tests had to be stopped after failure or 180 minutes, whichever came first. The same sized panel with very similar loading has been tested in the same furnace under the standard one way loading condition required by the fire test standard and reached failure after less than 20 minutes. In an initially undamaged multi-storied building subjected to fully developed fire and provided the columns didn't fail, two way action will prevail. However in the roof of a building, where the trusses are not composite with a concrete floor slab of where the supporting columns fail, one way action will prevail. (Clifton, p. c.)

These tests were done with concrete panels reinforced two ways with steel reinforcing bars or steel mesh reinforcing, and these tests were <u>not</u> large area, long span tests.

Mr. Clifton agrees that a 'two-way' composite floor panel system is analogous to a trampoline where the perimeter is supported laterally by an outside compression ring, 'in composite concrete the compression ring is built into the slab, "by virtue of the excellent compression resistance properties of good quality concrete. The reinforcement is not necessary for the compression resistance but is important for maintaining lateral stability of the slab panel" (C. Clifton pc), and its strength depends on a strong connections to the compression 'ring', and, a floor, strong in tension; just as a trampoline depends on a strong ring frame, strong springs and a strong center membrane having enough tensile strength, two ways to prevent ripping.

The reinforcing rods running two-ways are necessary for tensile strength and critical to effective two way slab panel action, since concrete has little tensile strength. The slab needs the compressive strength of the concrete to resist compressive stress in the top level of the slab, and when sagging into membrane tensile action, compression increases around the outside ring and holds the re-bars fast against the tensile forces. In firefighting the concrete protecting the steel rods from heat is sometimes spalled off by sudden water cooling as a hose stream is directed along the ceiling. Besides the type of aggregate the thickness of the covering of the reinforcing in the bottom of the slab appears to be important in protecting the steel.

Concrete, without sufficient reinforcing or deficient in strength due to varying ingredients and site added water, would offer little compressive strength to provide composite behavior to support tensile membrane enhancement upon deflection. Such concrete, possibly insufficiently reinforced with steel and/or weakened by site added water, and would probably crack up and just add weight to the suspension ties, increasing the pull-in forces.

It would seem that, over a large area, effective two way action in a floor assembly would require a stronger compression ring holding the trusses, than the spandrels in the Towers would offer, (especially along the long side of the perimeter wall). The WTC Towers the spandrel configuration, being composed of flat steel plates with strength only in the plane of the wall, would offer little resistance, against the pull-in forces produced by sagging, bar joists.
If one side of the floor connection to the perimeter wall failed, as apparently happened in the South tower's Northeast corner, one way action would surely prevail and would not inhibit collapse progression along the length of the floor.

Since the floors were designed to offer the main stiffening for, and between the outside bearing walls and were the main corner, anchor connections for these intersecting perimeter walls, with the weakening of the floors, the corner wall, spandrel connections would be the only effective cross wall connections. With just the 45 deg. champhered spandrels stiffening these cross walls, I doubt sufficient corner strength was available in the towers, to resist the pull-in forces, on the columns, upon floor buckling or sagging into suspension members. After the floors buckled and sagged, these champhered corner spandrels would have failed.

Possible Collapse Mechanisms – Tower 2- the South Tower

While watching the NOVA video of the collapse of the South Tower, I got the distinct impression that the East perimeter wall columns failed inwards rather than outwards as reported in the film. It seems likely that the intense heat of the fire along the East outer ring, office section and especially in the Northeast corner of Tower 2 (the South tower) affected the bar joists on several floors at around the 80th to 84th fl., and caused floor buckling and sagging into suspension which than pulled laterally, on the core columns and perimeter columns. The fact that these East wall, columns failed first, before the South wall perimeter columns which were most affected by the plane impact, is a good indication that the fire was more significant, than the aircraft impact, in triggering the progressive collapse. "If the collapse was inwards then this catenary action would have been a likely major cause of frame buckling initiation." (Clifton p.c.) However there are several possible mechanisms of failure which would cause the columns to buckle inward.

Dr. Usmani's new theory 'points to a compelling fire induced collapse mechanism that is unique to the Twin-Towers type of structure' and could explain the collapse of the South Tower. The extensive computer-based modeling done by Prof. A.S. Usmani et al. "show that the structural system adopted for the Twin-Towers was unusually vulnerable to a major fire." Thermal expansion of the floor trusses 'leading to compressive forces and buckling', 'dominates the behavior of the structure in the early stages' while 'the material effects(reduction of strength) becomes more important near failure.

Prof. Usmani's study found that, a serious fire on two or more floors would cause a buckling collapse of the perimeter columns. As the floor joists above the main fire floor and the second floor above that are heated in a two floor fire, the floors expand and push out the columns (about 15 mm -p.16). This however doesn't cause column buckling because the distance is limited and the columns, being connected to the joists, are held by the expanding joists from buckling outward. At this time the joists still retain 90% of their strength but they than buckle at low temperatures [400 to 500 deg. C, p.13], not from heat weakening the steel, but from the increasing compressive forces on these slender bar joists, as they expand against the columns and bend out of plane.. "If the floor system is very slender (as was the case for the WTC Towers), only a small amount of heating will make it buckle out of plane (because of restrained thermal expansion). "(p.29)

This low temperature 'restrained expansion' failure mechanism for long span, bar joists means that in any furnace test the fireproofing criteria adequate for shorter span bar joists, is probably inadequate for use in the long span configuration.

In a three floor fire, both the relatively cool 'fire floor' itself and the 'fourth floor above' the three fire floors, both of which are not much affected by heat, are first in tension, as the expanding heated floors are pushing out the columns. These three heated floors than buckle from the increasing compression, and thermal bowing and the columns being relieved of the forces rebound inward. The two cool floors switch from tension into compression from the increasing tensions in the deflecting heated floors pulling in the columns, and also buckle and than go back into tension assisting the pull-in. So in a two floor fire, 3 floors actually buckle and in a three floor fire 5 floors fail.

Dr. Usmani's et al studies analyzed a simple model of the WTC Towers for the effects of fire alone. They provisionally conclude "that these buildings could have collapsed as a result of a major fire event." (p.30) with no structural damage from an aircraft impact,

Another Collapse Mechanism

Indications are that the weak northeast corner of the South tower, had the most weight, and the hottest fire, because it received most of the wreckage from the plane and building contents which were swept there by the plane wreckage. The construction diagram of the corners (page 2-4, of the BPS study) shows five corner trusses, supported by one 'double header' truss, all bearing on one corner core column. This configuration apparently reduced the designed live, floor loads from "100 pounds per square foot (psf) over any 200-square foot area" to 55 psf at building corners." The diagrams also show numerous steel elements tying the main bar joists together, including 4 transverse, bridging trusses, perpendicular to the main doubled up bar joist trusses, extending the 207 ft length of the building, and the "intermediate deck support angles" between the main bar joists.

Northeast corner is where the stream of molten aluminum was seen flowing out a window, just before the beginning of the collapse. This molten flow could indicate that the floor in this corner was buckled or sagging into a catenary and possibly detaching from the columns, which could have been he trigger for sequential sag of the main trusses and the collapse. The molten metal possibly flowed down the sagging floor and accumulated on this weakest portion of the corner and increased the weight enough to snap the bridging truss connections to the North perimeter columns. Subsequently the increasing load, of the sagging floors was transferred, sequentially southward from truss to truss, and possibly also to the West along the North section of the outer ring.

There were only four bridging trusses per side and they were single rather than doubled up as the main bar joist trusses were. This buckling failure and consequent sagging of the main bar joist trusses would have created the suspension forces acting to pull in the perimeter wall columns. The fire could also have been dropping down, to the floors below, by way of cracks opening up in the concrete, or failure of the concrete panels and/or burning contents or fuel dripping through the floor, thus heating the floor joists, from below.

At least two floors or possibly three would probably be buckled and sagging into suspension since the floor pouring the molten aluminum out would have to be sagging, and the floor above the fire, that melted the aluminum (which melts at about the same temperature (1200 deg F.) that steel weakens, would probably be buckled also from the heat.

The 'double header' trusses (used as girders in the corners to support 4 main bar joists) were a week point and could have failed, either by heat caused buckling or sagging or breaking their connections to the columns. This 'double header' could also have failed because main bar joists that they supported expanded from heat and pushed the double headers off their seats and sheared off the bolts. Failure of this double header connection to the corner core column could trigger a 'rip' failure of the floor connections in two perpendicular directions along the core walls as the floors sagged and put forces on the joist connections sequentially. If the 'double header' disconnected at the perimeter wall, it may have started a 'rip' failure along the perimeter wall.

This floor buckling or sagging in the corner could also have transmitted sequential, bar joist buckling or sag along the middle of the East side floor; and also the North side since these 'double header' trusses were apparently directly connected to the North side transverse trusses. The same weakness existed in the southeast corner which probably had already failed due to the plane impact.

The Building Performance Study postulated a floor failure in the South Tower beginning at the southeast corner, possibly caused by the aircraft impact failure of core columns in the southeast corner, "followed rapidly by a collapse of the entire floor level along the east side, as evidenced by a line of dust blowing out the side of the building. As this floor collapse occurred, columns along the east face of the building appear to buckle in the region of the collapsed floor … causing the top of the building to rotate towards the east and [than?] south and begin to collapse downward." (BPS, p.2-35)

As this sag progressed both to the South, and possibly to the West along the outside ring, it is possible that the initial sequential joist connection (rip) failure impacted the floor below sequentially and induced catenary action in this lower floor. Since the core-columns were restrained from moving by their network of steel beam connections to the less fire affected, stable columns and floors on the West side of the building, the horizontal tensile forces acted mainly on the laterally unrestrained, perimeter-columns on the East side possibly pulling in the entire East perimeter wall. It would be a question whether these horizontal forces would break the joist-to-column connections first, or pull the columns in, breaking the weak, bolted column-splice-connections. The freestanding unsupported height of the, perimeter-columns would have been increased by the collapsing floor, and/or possibly by additional floors sagging into catenaries. The catenary forces, induced in the floor by the impact weight, deforming and buckling the floor joists, added to the floors own weight, were likely, enough to pull the entire line of East perimeter wall columns in, initiating the progressive collapse of the building.

According to structural engineer G. Charles Clifton:

> Close-up scrutiny of views of the impact regions (ie. Regions of significant structural damage) show little evidence of fire conditions generating temperatures of over 660 deg. C [about 1200 deg F (ie. fully developed fire conditions), except in the North-East corner of the South Tower… That corner of the South Tower was subject to fully developed fire conditions from immediately following impact to collapse, a time of 56 minutes. This fire was seen to produce streams of molten aluminum, indicating temperatures of over 660deg C. Quintiere et. al… estimate an average fire temperature of 900deg C over that region, which is realistic. The floor trusses within that region were passive fire protected with… three quarters inch of sprayed mineral fibre. This material was in the process of being upgraded to …one and one half inches. They have performed a heat flow / structural analysis on the floor trusses with …three quarters of an inch of mineral fibre and show that the estimated collapse temperature of an individual truss acting in a stand alone manner would have been reached after 42 to 59 (corrected to 56-94) minutes. The actual failure time of 56 minutes is [barely] within this period. Given that the impacts would have left the edges of the floor system adjacent to the impact region without the boundary conditions necessary for slab panel action, it is entirely possible that failure of insulated floor trusses in the North-East corner after that period of time was the final factor leading to collapse of the badly damaged system. (Clifton , "Causes and Lessons Learned", & Quintiere et al., Fire Safety Journal Vol. 37 p 707)

The NIST photo (Progress…p.39) shows a floor section on East face of the 83 floor, apparently disconnected from a line of columns and is sagging across a number of windows on the 82nd floor. This 'rip' disconnection, rather than the one described above, or in addition to it, may have sequentially progressed, along the floor, assisted by the 'deck support angles' and the 'transverse trusses'. When the floor, along with the office furnishings sliding down the tilting floor, impacted the floor below they may have set up sequential buckling in that lower floor which induced sufficient pull-in forces acting the columns to cause buckling.

Alternatively- if the floors expanding, buckling and sagging as in Usmani's theory, began on several floors and over a long length of these floors, either by restrained expansion or weakening of the steel, the tensile (catenary) forces would be spread over a large area of the perimeter wall and may have served to buckle the columns and snap the bolted column-splice-connections before the bar joist-to-column connections on any floor failed. Differential expansion of the steel, perimeter, box columns due to heat differences between inside and outside surfaces may have caused the thermal bowing, inward of these columns, and increased the compressive forces acting in the adjacent expanding top chord of the trusses hastening their buckling. As each truss buckled and switched from compressive forces to tensile forces acting on the perimeter wall it could have hastened sequential truss failure by transmitting the forces (the rebound effect) to each adjacent expanding truss through the spandrel connections. This sequential truss buckling could be transmitted two ways at once along the perimeter wall.

After a critical distance, the deformation out-of-plane (dishing in), due to the thermal bowing effects and the catenary forces pulling in the columns, the weight of the East perimeter wall and attached floors, above the bowed in section, would come into effect and assure continued column buckling, and wall failure. The column buckling may than have been assisted by being sequentially transmitted by the spandrel girders which connected the perimeter-columns, in the same manner as the bridging trusses could have helped propagate bar-joist consecutive failure.

I doubt whether catenary forces in the floor joists, on just, one floor could buckle the strong perimeter wall, before breaking the joist-to-column connections; since only one third of the columns, on any one floor, would be spliced and these steel box columns would be fully supported by the stable floor connections both above and below, unless of course these floors also, were also sagging. Prof. Usmani's most realistic fire scenarios do not show collapse for any single floor fire. (p. 13) The actual force developed by catenary action in such situations would have to be determined by testing. There are significant differences between the results of Mr. Brannigan's formula and Mr. Usmani's results.

Two Way Action

The WTC floor was apparently supported, in some areas, two-ways by intersecting bar joists and bridging trusses at right angles (a space frame). According to Mr. Brannigan; "When huge spans are achieved by rigid frames, trusses, or space frames, collapse can be sudden, general, and tragic." ('71, p237) I understand it as follows;

Say a fire begins in the middle of the floor. As those trusses expanded buckled and failed over a widening area, their weight and the self weight of the floor and live loads they supported would be transferred in increasing amounts to the remaining sound trusses surrounding the weakened area. A point would be reached where the remaining trusses would no longer support the increasing loads and complete failure of the floor would ensue even though they were not all heated. This failure would result in pull-in forces on the perimeter walls and core columns if the floor sagged into suspension. In two-way floors, if the spans were the same, I imagine that these catenary forces would be divided in half, between the trusses which were at right angles to each other.

Theoretically in accordance with Mr. Branegan's formula the longer bridging trusses used in the WTC would induce greater horizontal tensile forces than the shorter main trusses when buckling into catenarys. But since they would elongate more and fail before, and therefore be supported by, the main trusses (Quintiere et al.), the greater suspension forces would be developed by the shorter main trusses. I assume the concrete would break up and not add much rigidity which might attenuate these forces. Of course if one side disconnected from the frame, as apparently happened in The South Tower, than the full one-way catenary forces would be acting on the remaining connections.

If, the joist-to-column connections failed this would allow the floor joists to drop either simultaneously or sequentially to the floor below imparting greater catenary forces on to the columns which had just shed some of their lateral support by the floor failure. Of course additional floors above or below could also have been sagging due to heat and their tensile forces would be added to those already pulling in the East perimeter wall. All this should be investigated by long span testing or computer modeling based on such testing.

According to the NOVA video, after buckling of the entire East perimeter wall, along several floors the top portion of the South tower first began falling "away from the impact [South] wall". The angle of failure, however, is difficult to gage due to differing camera locations. The top portion of the South Tower apparently first hinged on the intact robust, West perimeter wall. The South wall, damaged by the plane, was crushed or buckled after the entire East wall buckled inward. The weight and rigidity of the top portion of the building's perimeter wall, - a vierendeel truss (a truss composed of rectangles, instead of triangles) - along with those floors still attached, caused the entire outer perimeter wall to collapse as a unit. It first apparently hinged on the West wall, than being bumped over to the South by the still intact North perimeter wall, leaving the core section intact, for a time, as the outside ring collapsed around it. The core too, then disintegrated, from the undermining forces of the collapsing perimeter frame.

This perimeter wall failure would also transfer its weight and the attached floors to the core columns, through membrane action and the hat truss on the top floors; but since the NOVA video showed the core, or a large section of it, remained standing for a time, after the outside ring fell, the hat truss must have been ineffective in supporting the perimeter walls and the floors probably hinged at their connections to the core and perimeter columns. These connections must have than sheared off as the perimeter wall fell.

If the core columns had failed first, as postulated by some engineers, the core would <u>not</u> have remained standing for a time but would have preceded the perimeter walls down, probably pulling and buckling them inward during the collapse. Therefore, at least, in the South tower, I would think that, the pull-in forces, caused by the long-span bar joist, floor trusses buckling and sagging from the heat of the fire, or impact of falling floors, pulled in the East perimeter wall initiating the progressive collapse of the building.

Possible Collapse Mechanisms - Tower 1 – the North Tower

The North Tower (Tower 1) collapse scenario is more difficult to understand, and several mechanisms are possible. The fire was burning longer before the collapse, - 1 hour and 42 minutes vs. 56 minutes for the South Tower-, raising the possibility that the core column failure, from heat, could have been the primary or a contributing cause.

Most of the engineering reports, which I have read, cite core column destruction by the plane impact and subsequent failure of the remaining core columns by heat weakening, as the major North tower collapse cause. According to Clifton… "The impact damage- not the severity of the fire – I contend is the principle cause of the ultimate collapse. … and… The likely influence of the fire in the time from the impact to collapse would have been to progressively weaken the residual vertical load carrying capacity of the remaining core columns…" (See Clifton, Dec. 01). According to the Building Performance Study, "This suggests that the collapse began with one or more failures in the central core area of the building."(P.2-27)

Less building weight would have been involved in the North tower, since the collision and fire occurred on a higher floor, floors 93 to 98 vs. 78 to 83 in the South tower. This however, probably also meant lighter weight columns which would be affected sooner by heat. More perimeter columns were taken out on Tower one's North face, about 35 vs. 30 in the South tower; although the plane was traveling slower when it hit the North tower; about 480 mph vs. 560 mph. The amount of fireproofing insulation removed by the impacts is under contention and since the 3hr furnace tests are probably more severe than the actual fires at the WTC were, the columns should have lasted much longer than their actual designed endurance times, if most of the fireproofing remained intact after the impacts. These columns would have outlasted the fires which would have burned out in 2 to 3 hours, per floor as estimated by Prof. Quintiere et al. I believe these burnout times should be experimentally tested in typical office occupancies so that an endurance time for structural members, with some added safety factor, can be determined for code development. It would seem that in buildings that have illicit openings in fire barrier floors, super heated smoke can accumulate on the upper floors and be heating the structure for as long a time as it would take for each floor added together to burnout.

However, the reports are that Tower 1 had thicker fireproofing on the bar joists, which could have added to a delay, in floor joist failure compared to the South Tower. An analysis of the fireproofing by Prof. J.G. Quintiere et al., finds that the failure times agree with the amount of insulation on the bar joist trusses. "The insulation thickness and the difference of …3/4 inches and …1 and ½ inches between the two towers appear to have been the root cause of the collapses." "Neither tower, he found had fireproofing thick enough to withstand the fires blast furnace intensity for two hours, which is considered the minimum needed for those on the upper floors to escape the towers." (Washington Post 6\25\02). His "structural failure model is … based on compression buckling of the truss rods due to a reduction in the Young's modulus."

It seems we also have to consider truss failure, as a triggering mechanism in the collapse of Tower 1. His team calculated the average time to failure of 115 minutes for the North tower and 75 minutes for the South tower. They actually failed at 104 minutes and 56 minutes. "It's the only calculation I've seen that has any correlation with events" Dr. Quintiere says." Also, "The fact that the buildings stood as long as they did suggests the insulation remained intact on many structures, [despite the impact forces]." (FSJ Vol.37+ added corrections).

. Even though some of the core columns were possibly damaged, with largely intact insulation further weakening by heat caused, creep buckling would not have occurred until well after the actual collapse times.

There would have been little catenary action on the North side, since a large portion of the floors were destroyed or detached from the perimeter wall during the aircraft collision and there appeared to be little fire in this area. Eyewitness reports from the South tower indicate heat radiating through the windows from the North Tower as the fuel explosion occurred. This indicates a large fireball must have been emitted, indicating widespread ignition in the South, outer office ring. Still pictures, of the South side of the North Tower, taken around the time the second plane hit the South Tower, shows flame over a long section of one or more floors on the South side and smoke out the windows of several others, in the North Tower

As in the South Tower, an intense fire, in Tower 1 seems to have been burning on the side away from the impact. According to G.C. Clifton, "It is likely that the temperatures in some parts of the impact region would have exceeded 700 deg C. (1300 deg F.), especially on the South side of the North tower." (#1). Prof. Quintiere et al. reports "Further, the images of the collapse of [Tower 1] seem to show a burst of flame and smoke outward just before the collapse. This in our view corroborates the idea of a generalized floor failure. The collapse of a floor would cause the smoke and fire to be pushed out of the building from the space below as video images seems to indicate" NIST reports a sudden line of smoke at the 92 floor along the North side and smoke also at various locations on the 93,95 and 96 floors nine minutes before collapse of the North Tower.

The fact that in the NOVA video the antenna on top of Tower 1 seemed to begin falling before the North façade, was given as an indicator of core failure preceding perimeter wall failure (BPS,p. 2-27), but this movement, if it happened, appeared to be very small and the appearance could have been because of the building, first, leaning slightly away from the camera location, or the South wall failing first, along with the core and dropping or tilting before the North perimeter wall, and not, because the core was failing independently of the perimeter walls. Buckling of the core columns on the South side could also cause this lean. The NIST progress report indicates that the North tower began to lean to the South 7 minutes before the collapse. That's two m9inutes after the appearance of smoke which could indicate multiple floor failure. The upper portion of the building (the perimeter frame) appears to fall as a unit with the antenna and thus the core; this after total column failure along a line of floors at about the level of the plane entering the North face or slightly higher. To postulate that the antenna fell about a meter immediately after the impact of the plane is without evidence.

Since the robust outside frame and probably the core columns were not much affected by heat, the only way you could have instantaneous wall failure around the building is if a floor or number of floors collapsed and removed lateral support for the remaining columns or they were displaced and buckled laterally by expansion or tensile forces, pulling in and/or pushing out the columns.

So we have to explore the possibility that the bar joist floor trusses buckled and/or collapsed on South side, which is the long span side, of the North Tower in much the same manner as in the South tower. This would be a major indication of fire induced collapse, since the South side of Tower 1 was the side certainly least affected by the plane impact. The 'fire was moving downward 12 minutes before collapse' (NIST Progress…, p37) This fire dropping down could have been an indication of a floor or floors buckling from restrained expansion and dumping burning contents through the damaged floor, on to the floor below. This buckling would have probably occurred near the outside windows since the fire would be hotter here from receiving more air and the longest truss rods which connected the bottom chord and the top chord, attachment points of the joists-to-columns were at this location and these end truss rods would have elongated a greater length here when heated and assisted the buckling at this location.

If, a series of bar joists buckled and sagged into suspension over several floors the concrete and burning contents falling through the damaged floor would have impacted onto each floor below. Any accumulated sprinkler water on each floor would also have flowed towards these openings in the floors near the windows and served to overload the floors below possibly inducing more joist buckling, assisting column buckling. This water would have accumulated on any intact floor without openings and spread out and possibly overloaded an entire floor, causing buckling or collapse.

From the study of videos, NIST reports a line of smoke across the 92 floor, simultaneously with smoke out various windows on the 93, 95, and 96 floors three minutes after the fire was seen moving downward and 9 nine minutes before collapse. This expulsion of smoke could have been from the falling floors forcing the smoke out, as they detached from the perimeter or core wall. There is no mention of South perimeter wall column buckling. Absent such buckling we must assume multiple floor disconnection.

Possible Triggers for Multiple Floor Failure;

1. The build up of weight along the perimeter wall as each floor buckled near the windows and the furnishings possibly sliding down the floor into this buckle could have pulled the columns in a certain distance until the bent ends of the trusses were almost hanging straight down. This would have limited further column bowing in as the resultant forces on the perimeter columns would have been close to vertical. This kink in the floors loaded with extra weight could have put extra downward strain on the bolted and welded joist-to-column connections, causing failure of these connections, setting off a series of rip failures and impact detachments along the perimeter wall without buckling the columns. This could have sequentially detached a number of floors while leaving the South perimeter wall and core standing but without their lateral support.

2. Or when a corner floor section on the South side, began to expand or sag, and a "double header' girder possibly detached, starting a 'rip' failure of the floor along the core section. If one floor started detaching along the core, the desks, chairs and file cabinets could slide down the slanted floor and impact the floor below possibly accumulating enough kinetic force to begin a sequential rip failure in the floor below. With the impacts of the detaching floor traveling both ways around the outer ring, its possible; you could have had rapid sequential floor failure along the South core wall and possibly around the building, affecting five or more floors, especially with the added weight of furnishings and some sprinkler water collecting along some floors.

3. Or for one of the reasons above, causing a sag or buckle in the floors, the tensile forces in the floors pulled a line of core columns outward on the South side either sequentially or simultaneously. This could have happened at a line of core column splices which failed and let down a number of floors on the South side and started failure of the core columns causing the lean of the North Tower to the South. This lean was evident 2 minutes after the appearance of the smoke, which is suggestive of multiple floor failure, 7 minutes before the collapse.

Or this lean could have been caused by the remaining columns beginning to bend or bow possibly assisted by one or more core floors being displaced by the lateral forces of the collapsing outside ring floors.

Some portion of the South perimeter wall could have buckled in along with the multiple floor failure, but this apparently would have been seen and I haven't heard any reports of this. The absence of reports of buckling of the South wall would favor the theory of floor detachment over wall column pull-in.

With the destruction of 35 North facade columns by the aircraft collision, this potential multiple floor failure on the South and partially on the East and West sides would have removed practically all of the lateral support for the core and perimeter columns over those floors. Only the hat truss and remaining upper floors would be holding the outside frame and core together laterally picture ,hat truss BPS, p.2-10-

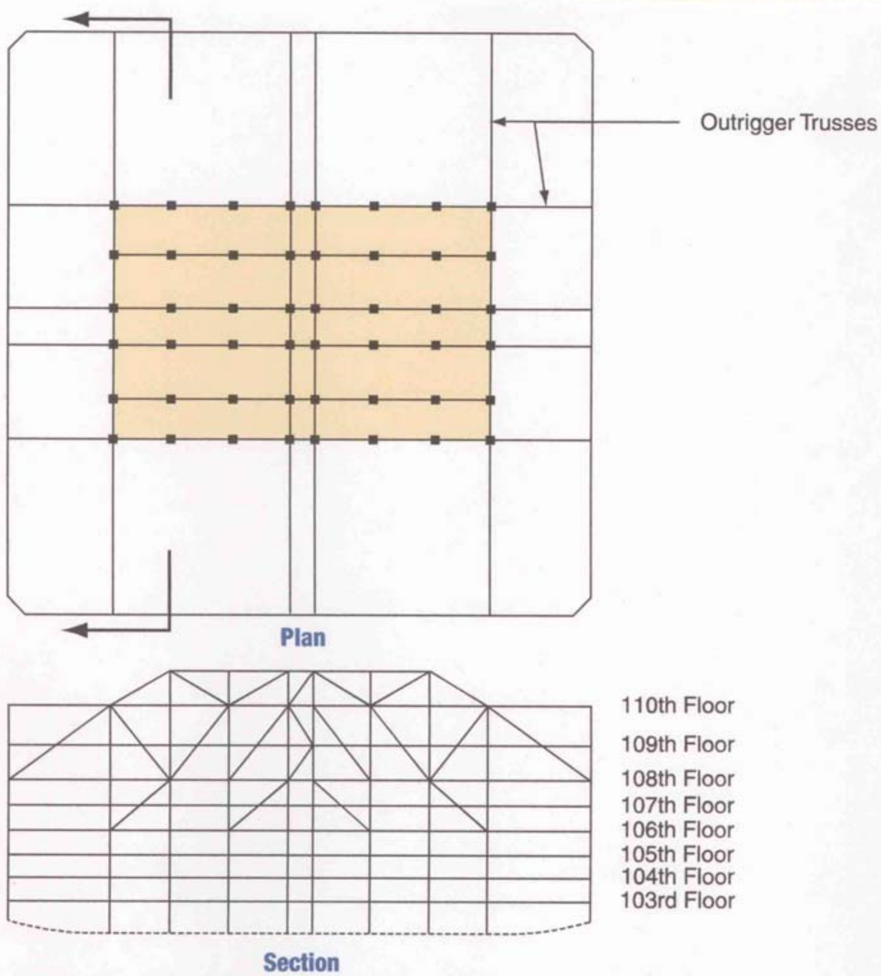


Figure 2-10 Outrigger truss system at tower roof.

Or, after the failure of so many outside ring floors in the North tower, we could also hypothesize any remaining attached floors on the South, East or West side putting horizontal forces on the core floors which could pull the core floor plates out of alignment precipitating the buckling of all the remaining columns along with the perimeter columns. If fires were burning on different levels on the East and West sides, putting some levels of floor joists in compression due to heating and some different levels in tension due to subsequent buckling and sagging on opposite sides, this could serve to pull and push different level core floor plates in opposite directions forcing the columns into the classic S buckling shape.

Any of these situations could buckle all these remaining columns, simultaneously, along these floors, and bring the building straight down. I don't know at this point how or at what levels the core columns were spliced but this also may have been a factor. According to Clifton, "This effectively instantaneous failure occurred I am as sure of it as I can be."

The remaining upper floor joists and hat truss would have contributed to pulling down the core section, as a unit with the outside walls. The perimeter columns under this scenario could have buckled inward, effectively telescoping the top portion of the tower into the bottom. Of course after the entire upper portion of the building started down the momentum would have "sheared off the lower floors …so quickly that the inwards and outwards forces on the perimeter frame were near equal. Evidence of this is seen from several sections of perimeter frame over 50 stories in height remaining standing for some 1 or 2 seconds after the core and floors had fallen." (Clifton p.c.). This final collapse stage where the top portion of the building fell intact as a giant falling wedge peeling the perimeter walls off, may have been the basis for the report by Malott of a "bulging ripple going down the outside skin in advance of the collapsing floors." The spandrels could have acted as cleavers shearing off the joist-to-column connections.

Further Explanation

In my estimation, 'if' the core section failed first from buckling, as postulated by most engineers, the antenna would have descended well before the perimeter walls and the outrigger struts in the hat truss (BPS p. 2-10) would have pushed out the perimeter wall at their attachment points on the 108th floor due to the angle that they were set.

Clifton assumes some core columns were severed by the plane and the core section dropped and the outside ring floors, which would have sagged, would put these core columns in tension, and this tension "would have traveled up the columns to the top" In fact, if the core came straight down all the floors would have sagged the same amount, since all the outside ring floors would be constrained to sag the same amount by their connections to the core columns. All floors would share the weight of the core equally, and all would theoretically have "spread that extra load around all the connections between the upper floors and the perimeter frame." an equal amount. Theoretically their connections would be under the same tensile stress, but the top floor perimeter walls which were being pulled in by this force, would 'bow in' a greater distance because they had less lateral strength, and would lean in more at the top floors.

Because of the angle of this lean, the top floor connections to the perimeter wall would actually be under less tension than the lower floors, and the tension would increase as you went down from floor to floor. The greatest tensile forces pulling in the perimeter wall would be at the lowest floor above the severed core columns. This lowest floor over the impact region would detach from or buckle the perimeter frame first, not the top floors. Actually this scenario would match the facts of the collapse better than 'detachment of the top floor' (Clifton) since this increased force would more likely buckle the perimeter walls at this lower level. However, as described below, the core falling would buckle the East and West perimeter walls, before the South wall, in which case the building would hinge on the intact South wall and tilt to the North. But this didn't happen, and thus suggests core failure was not the initial cause of the North tower collapse.

Under the situation where the core was dropping, the pull in forces would actually be greater at the short span sides of the outer ring. Because of the shorter lengths of the joists at the East and West sides, as the core moved down, these, side joists would come under more tension than the long span sides. This, as I mentioned, is because of the geometry. These short side floors would put the perimeter walls under more pull-in forces than the long span sides as the core descended. The East and West sides of the perimeter walls would buckle in more than the North and South long span sides. I assume this would have been noticed before the collapse. The East and West side floors would have been the first to detach at the connections, or buckle the columns at the lower, severed column, floor level.

Once the core started to descend we would be no longer talking about 'the vertical load carrying capacity of the perimeter frame'. The resultant forces would contain large horizontal components which are somewhat attenuated by two-way action but none the less extant and cannot be disregarded.

If we, for example, postulated removal of the core and extended of the existing floors, as built, over the whole one acre floor areas, I believe, there would be immediate sagging of these floors into suspension membranes because of the weight of the concrete and steel joist stretch out. Just as in the Madison Square Garden roof where there is a strong compression ring resisting the tensile forces of the membrane, the Tower's perimeter walls would serve as a compression ring and the floors a tensile membrane system exerting large pull-in forces which the twin towers were not designed to handle. Since there would have been little resistance to pull-in from the spandrel girders, the perimeter walls would have deflected and crumpled inwards to a noticeable extent, as the core descended.

If, as some theories surmise, the top floors detached and fell first, as each floor fell it would have expelled quantities of smoke out any available openings such as the broken out windows, but we only see this from the 96th floor down to the 92nd floor. Rather than an indication of top floor failure, the smoke seen being expelled from the top floor during collapse could have been pushed up the open shafts to the top floor and been expelled from the open louvers around the top floor perimeter, as the tower began collapsing.

Also, judging from what happened in the South tower, if, in the North tower, the core section failed first, the building would have leaned over to the North side, as it collapsed, since the South perimeter wall and its floor connections would have been intact. This south perimeter wall would have acted as a hinge and tipped the building to the North, since all the major column damage from the collision was on the North side. The perimeter frame, however, apparently failed all around the building at about the same time.

If you postulate core damage as being a precipitating cause of collapse, you would think most of the core column damage occurred on the impact side. This would shift the weight of the core to the North and when these North core columns failed, the weight of the floors they held would be shifted to the damaged North perimeter wall, further loading it. I would assume that the building and antenna would tilt however slightly to the North and not the other way, especially since the engineers see no damage to the South perimeter wall.

At this stage of the investigation, I have to conclude that, there is a good probability that the fire, acting to heat the floor trusses in the South side caused their failure which, (triggered by buckling forces or catenary action), detached multiple floors leaving the remaining core columns and the perimeter walls, at these floors, without lateral support for a few minutes. This allowed simultaneous buckling of all columns on these levels somewhat as described above continuing the progressive collapse of Tower one.

Smoke Explosion- Another possibility?

The higher floors were receiving most of the super heated gasses rising up the damaged elevator and stair shafts and other vertical openings such as any un-fire-stopped pipe or wire runs, air conditioning ducts or cracked openings in the concrete floors. These fire gasses could have accumulated and heated the entire ceiling area or the truss voids of one, or more floors, starting a softening and sagging of the joists. Or, more likely, after filling a floor (mushrooming), these heated gasses could have exploded, triggering the initial floor collapse. Smoke explosions happen, at times, in unventilated spaces at serious fires. (See "Backdraft", WNYF, by Chief Norum)

If the collapse did begin, simultaneously, around an entire floor, (outer ring and possibly the inner core) of Tower 1, this coincidence would suggest an explosion or rapid combustion of flammable gasses, such as carbon monoxide or vaporized jet fuel suddenly, over-pressurizing the entire area. Incomplete combustion, due to lack of oxygen, in the main body of fire, in addition to producing these flammable gasses, may have been another reason the fire temperatures in general not being any greater than an average fire. According to Clifton "The observed fire behavior points to temperatures in the building not being particularly severe – say no more than about 600 to 700 Deg. C. [1100 to 1300 deg. F] Possible reasons for this may involve the coating of combustible material in dust from pulverized concrete and gypsum board and the volatility of the aviation fuel leading to large amounts of fuel being pyrolised but not burnt in the interior of the building."(Clifton, "Elaboration…" p., 6) It is also possible that some portion of the sprinkler system was working bringing down the temperature. This would have lessoned the chances of backdraft.

Pyrolysis involves thermal decomposition in the absence of oxygen. In a large area building fire the high heat may be distilling off (generating) more flammable gasses from combustibles than can be burned, since the available air is being used up in combustion, and fresh outside air is forced away by the expanding fire gasses. These flammable vapors and gasses, produced by heat but unburned, can migrate to remote spaces above the fire, due to rising convection currents, where if they attain the right mixture with air and are hot enough, will burn and possibly explode. "A room or area requires only 25 percent of its space to contain the explosive mixture for the entire area to explode."(Dunn, "Safety and Survival…" p.138) This is another of the reasons why upper areas in fire-buildings are ventilated, if possible, by the Fire Dept.; to remove these heated toxic gasses, prevent fire extension and possible smoke explosion.

The overpressure produced by rapid combustion can vary from low pressure as in a flashover, to severe as in a backdraft. I believe the force of a smoke explosion could, be sufficient, to damage and induce catenary action in the heated long-span floor joists over an extensive area. The postulated overpressure in Tower 1's upper floors and/or bar joist voids may have been strong enough, with the heat weakened bar joists, to start the floor collapse. However, if you carefully observe the NOVA film of the collapse in Tower 1, you can, notice a sudden plume of black smoke over the roof and other areas expelling smoke, just before the progressive collapse becomes evident. I don't know where this smoke came from since I haven't yet seen any video footage of the South side of Tower 1, but it could have come from the South side and be an indication of further floor failure, or buckling of the South perimeter wall or a smoke explosion or both.

As the truss failure theory gains credibility backdraft seems less probable, except possibly as an initial trigger for a floor collapse,.

.

Effectiveness of Full Automatic Water Spray (sprinkler) Protection

According to Chief Dunn, “The only real fire protection for a commercial or residential high-rise building is an automatic sprinkler and smoke-removal system to vent the smoke after the sprinkler extinguishes the fire.” Mr. Brannigan comes to the same “…inescapable conclusion that full automatic sprinkler protection is vital to the safety of occupants of high-rise structures.”(Brannigan, p370) “However, if the fire originates in or penetrates the [truss] void, the sprinklers will not be in a position to control the fire”. (Brannigan.p548, 01)
In my opinion, total sprinkler protection including the truss voids, if it had been installed and remained intact, would have provided enough cooling of the remaining protected steel to delay or prevent total collapse at the WTC fuel fire. When the B-25 bomber hit the Empire State building, “The operation of five sprinklers in this building gave the alarm and held the fire in check until firemen arrived.” (NFPA, “Fires in ‘Fireproof’ Buildings”, 1950). An intact sprinkler system certainly would have reduced the smoke and heat output to a more manageable level possibly saving many more lives. (Full automatic sprinkler protection means every area and room and every void space on every floor is covered by the discharge pattern of a sprinkler head.) The importance of early fire control, to save lives and property, by reducing fire and smoke production, and by prevention of collapse is generally not appreciated even by engineers.

Drawbacks of Partial Sprinkler Systems

Partial sprinkler systems can be troublesome is; if the fire in the unprotected area gets out of control and cannot be cooled quickly, the heated gasses produced can ascend up elevator shafts or other openings such as un-fire-stopped pipe and wire shafts or poke through openings, to remote floors above the fire and set off the water spray heads there, possibly overloading floors and playing havoc with elevator control systems, stalling elevators in the shaft ways.

Since the areas above an uncontrolled fire may be dangerous to enter because of developing smoke and heat conditions, the Fire Dept. may not be able to shut off the sprinkler water branch line valves. If in the WTC the steel bar joist floors have begun to sag from the heat, water would tend to accumulate in the depression, created by the sag in the floors, further overweighting the floor, hastening collapse. Again full coverage by sprinklers will, paradoxically, mitigate this problem by reducing or eliminating the production of these super heated gasses from unsprinklered areas which could set off remote sprinkler heads. On-off sprinkler heads and floor drains or scuppers to drain the water may also help. In spite of all these problems, “Sprinklers are the core of fire safety for the occupants of high-rise buildings” (Brannigan 1992, p502). I might mention that water spray, sprinkler operation in most of the WTC buildings was ineffective at controlling the numerous fires. This condition is usually due to poor inspection, testing and maintenance, & repair of these water spray systems and instillation of partial sprinkler systems, but at the WTC it could have been because of destruction of the delivery system at the points of impact.

Air Conditioning

Central heating, ventilation and air conditioning (HVAC) systems could have affected the building stability and smoke contamination on remote floors in several ways. According to Former Fire Commissioner and Chief of Department, John T. O'Hagan; writing in his book High Rise / Fire and Life Safety;

> "The air-conditioning system increases the flow of air and of oxygen to the seat of the fire, thereby increasing its rate of development and its ultimate severity. The return portion of the system recirculates smoke and contaminated air on the floors above and below the fire increasing the life hazard, complicating the evacuation and rescue problem, increasing the difficulty of locating the seat of the fire and delaying the actual extinguishing operation. This allows the fire to increase in severity and extent. The undivided ceiling space which is utilized as a plenum for the collection and direction of the recirculated air to the return shaft is also an effective medium for the transfer of heat to the remainder of the floor area."(p132)

This configuration,- using the truss voids as a return space for air movement back to the mechanical equipment fan rooms, used at the WTC would also negate any fire protection that the ceilings afforded the steel trusses, thereby hastening their buckling, sagging and eventual collapse. Unless the ceilings were fire rated and the air conditioning registers had automatic dampers, even shutting down the return fans would not stop superheated gasses from entering the truss voids. The ceilings were not fire rated and accordingly Prof. Quintiere et al. gave them credit for 10 minutes protection for the bar joists in his fireproofing calculations. "The ceiling was almost universally a suspended non-fire rated ceiling… If not immediately destroyed this membrane would likely retard the direct heating of the insulated truss by about 10 minutes, an estimation based on experience."

Deputy Chief Elmer Chapman gave criteria for an effective HVAC smoke control system. He writes, "A positive means of smoke control should be provided in all high-rise buildings…" and he gives 19 necessary criteria, for such a system, including "Any proposal that allows a smoke control system to ventilate a fire building before the application of a suppressant is wrong… [also]… There is no possibility of designing an economically feasible smoke control system that would be capable of handling the tremendous volumes of smoke that would be generated in an unsprinklered building. Six hundred square feet of burning hardwood will fill 200,000 cubic feet of space with smoke to a degree that is beyond human endurance. The pyrolysis of plastic material will result in the production of 500 times as much smoke as hardwood, in addition to the production of toxic and volatile gasses". (WNYF, p.19, 3rd Issue '83)

Collapse Inevitable

Dr. Asif Usmani a structural engineer at the University of Edinburgh School of Engineering and Electronics reported that the Towers seem to have been "unusually vulnerable" to a major fire. His computer analysis assumed an intact building with no aircraft damage. "There was a vulnerability in the design of the structural system. It is not the materials. It is about the design of the structure" (Reuters June 4, 03)

In my estimation, in addition to the design deficiencies in the use of lightweight, long-span, steel bar joists, the Trade Center Towers, given the large open areas, undivided by fire walls, the deficient 'fireproofing' on the steel, the weak column splices, the probable, numerous, poke-through openings in fire containment walls and floors, and possibly weak concrete floor panels with no apparent effective reinforcing, any large area two floor, uncontrollable fire in these buildings would have produced the same result, total collapse.

We hope NIST will retain continuing funding to be able to continue the important work to determine parameters leading to collapse and effective corrections and building codes in both existing and future high-rise buildings

Building Codes

For some arcane legal reason the Port Authority of NY State and NJ did not have to comply with the New York City Building Code, and Fire Codes. Since New York City has been the premiere skyscraper capitol of the world, the City Building Code regulations and enforcement capabilities have evolved out of the many fire disasters over the past 200 years.

Building and Fire Code regulations and procedures are of critical importance for life safety and property protection. If, the Port Authority had to submit plans and get plan approval from the City before starting construction, it would have been subject to plan review by experienced Code experts and ongoing inspections, during construction, by experienced Building structural engineers and Fire inspectors who had the power to stop the job until construction deficiencies were corrected. The Port Authority would have had to receive a final inspection and certificate of occupancy before opening the building. If this plan review had been done, by experienced engineers, the design, construction and active and passive fire protection would have been changed and many more lives could have been saved. Any NYC Chief certainly would have objected to long-span, steel bar-joists being used, and sprinklers being omitted. (Sprinkler systems were not even built into the original Trade Center Towers,)

All buildings built in the City should, at least, have to follow the City Codes and be subject to design scrutiny and compliance inspections during construction and afterwards, and certificate of occupancy inspections, before opening the building to the public. The Port authority had a 'policy' to comply with City Codes, but still there were serious deficiencies in building design, steel protection from heat, exit-ways & enclosures and fire stopping, which would have been detrimental in any serious fires in these buildings. In the N.Y. City code, "Controlled Inspections" are allowed whereby engineers, hired by owners and builders have authority to inspect and sign off on building inspections which were formally performed by City Building Inspectors. I realize this is to fix responsibility where it belongs and reduce the workload of the City Inspectors, but on site, check up inspections of records and violations themselves should be performed to assure compliance.

Recommendations, Building and Fire Codes

Height and Areas

The N. Y. City Building Code (See table 4-1, 4-2, p.63, 64 of Ch 27) places "no limits" on height and areas in many building categories. This is ludicrous, and needs to be reviewed.

Since the effective coverage of hose streams is limited, at large area fires, and sprinklers are sometimes inactivated due to repairs, the area of open floors, should be limited by fire containment walls (2500 or possibly 5000 square feet has been suggested as a maximum), in order to keep fires to controllable size, in the upper floors of high buildings. Sprinkler systems, for such large areas, should be redundantly designed for dependability, even in low rise buildings. The down time for sprinkler system repairs would be eliminated or lessoned in redundant systems.

Construction

Any and every critical element and its fire protection may be important in maintaining the integrity of the entire building at a serious fire. Since this situation is compounded as a building's height and weight is increased, columns, girders and beams and walls and floors should be strengthened with an increased factor of safety; and redundancy of protective systems increased accordingly, to take care of unexpected emergencies in high-rise buildings.

According to Prof. Quintiere a reasonable factor of safety should be applied to fireproofing insulation endurance times in high-rise construction. Buildings over a certain size should have maximum fire resistance no matter what the occupancy. The early City Building Codes of the 1930s to 1960s required more reliable fireproofing and greater factors of safety in high-rise construction, than exist in today's codes.

NIST is conducting full or real scale representative tests and computer modeling for the WTC long span, bar joist and steel box column configuration and we await the results. Pull in forces should be measured for buckled or sagging trusses and whether catenary action is induced by the impact weight of a floor falling from above.

Full-scale fire tests for 'long-span', I beam floor assemblies with sprayed-on "fireproofing" should also be conducted to determine their actual fire rating (endurance time), and possibly what the effect of removal of sections of sprayed-on fire insulation would have on the collapse resistance and catenary action of such 'long span' steel girders or beams. These tests should include tests using suspended ceilings as HVAC return plenums for the heated gasses, to determine the effect this configuration has on I beam, bar joist and Q deck supported, concrete floor failure.

Note: According to Charles Clifton HERA engineer; "Any testing of joist floor systems should simulate the as built support conditions so that two way action can occur. This will involve testing more than just a one-way slice of the building and saying it is representative of two way action, because it won't be. The key component to look at, in my opinion, is the connections between floor and supports. These must be designed and detailed to accommodate the expected inelastic rotations and pull-in displacements."(Clifton p.c.)

If these tests prove that these, long-span, lightweight, bar-joists contributed to the early failure and progressive collapse of the building, than long span, steel bar joists should be prohibited for floor construction in any public building of any size. Since this early failure problem would seem endemic, in any unprotected lightweight steel joist system, such as steel C joists, all unprotected lightweight steel joists should be furnace tested for safe collapse resistance in floor construction.

If this type of bar joist floor system fails the tests, I believe a survey and re-assessment of all existing buildings which use long-span, steel bar joists should be conducted, in order to consider rebuilding them, using conventional steel framing methods.

Since it is impossible to evacuate a high-rise building rapidly, and since long span floors (including I beam supported floors) are inherently weaker than short span construction and would fail sooner, impact load tests to determine their progressive collapse potential would be productive. A floor and its connections and supports should be built strong enough to withstand the impact weight of two or three or more, fully loaded floors dropped from above; water weight from fire suppression activities should also be considered. This is in order to prevent or arrest a progressive collapse. This has already been corrected in the N. Y. City code.

Include tensile catenary action as a design requirement in building codes, for steel beams, joists and floor systems, to ascertain and compensate for their interactions with columns and walls

Tests should be conducted to determine the optimum and maximum safe spans for any particular beam and column or wall system subject to possible thermal buckling or catenary action, under fire or collapse conditions.

The effect of an ordinary natural gas or smoke explosion on such long span floors should be determined, and compensated for.

Columns should be designed to support their loads even with the collapse of several floors. Steel column end splices and beam splices should be required to be as strong as the columns and beams themselves.

Because of the devastation caused by a tall building collapse, columns and beams in high-rise construction should be protected by robust, impact resistant, thermal insulation imparting an endurance time of as long as a fire could possibly take to burn itself out, with a factor of safety to compensate for unexpected events.(Quintiere et al.)

I support architect Malott's and Chief Dunn's recommendations to encase columns and beams in concrete or masonry for protection rather than using current, ineffective, spray-on, fire insulating material. Impact protection for the fireproofing such as steel jackets, or wire mesh reinforcing such as wire lath and plaster could be tested to develop effective substitutes to prevent flaking off of the protection.

Impact tests, such as, the 60 psi, nozzle pressure, solid water stream test for 10 minutes, after the fire endurance tests for walls and floor assemblies and columns, as required by the early City Codes, should be re-instituted.

Evidently the impacts of plane parts or the fuel air explosion or the displacement of the building, by the aircraft momentum, destroyed some of the wall enclosures of the stairways and elevator shafts, and cut off escape from above by jamming exit doors in the frames or filling the stairways with debris and heated toxic smoke. Elevators were also disabled due to shaft destruction and flaming jet fuel, pouring down the shafts. Tests should be developed to determine whether the overpressure load of a fuel vapor & air, explosion alone, or the impact of a hose stream, could affect the integrity of the "shaftwall" gypsum board, enclosing the stairways, and elevator shafts. If an ordinary natural gas or smoke explosion, or the impact of an interior or exterior hose stream affects the integrity of stairways, exit corridors or elevator shafts than then this type of "shaftwall", gypsum board construction, should not be allowed for such use in any public building. As building heights increase more effective protection for exit way enclosures such as reinforced masonry or concrete should be required throughout.

The aircraft, impact momentum apparently moved the buildings several feet, racking the walls in the central core thereby deforming the door frames out of plumb, binding some exit doors in their frames. (See: Accounts from the North & South Towers, & Locked Stairways….) In the South tower the off center impact apparently twisted the building shearing off ceiling panels in remote parts of the building. (Nova report) This suggests inadequate diagonal bracing throughout the core areas of these buildings. Plan examiners should check for adequate lateral bracing. "When moment- resisting connections are not provided in a building, diagonal bracing or shear walls must be provided for lateral stability." (BPS, p.A-17)

The 'shaftwall' and other drywall gypsum were dislodged in numerous places by the impact loads, the fuel air explosion or the building shifting. This suggests, besides the fragility of gypsum board, that the means of drywall attachment was possibly inadequate. Tests should be conducted on the particular method used to attach this drywall to determine adequacy.

"The building code should require greater thickness of concrete in floors in high-rise buildings.
The four inches of concrete over corrugated steel used today: cracks, sags, buckle, warps, during most serious high-rise fires and must be replaced. Floor construction is the weak link in today's high-rise buildings."(Chief Dunn, DOB Task Force Hearing, Aug. 02)

Reinforcing in concrete slab construction should be required and designed provide two way action as in the Slab Panel Method developed in NZ by HERA structural engineers.

New, easily applied, fireproof material should be developed, for code approval, to serve as fire-stopping to re-plug, poke-through holes in fire walls and fire barrier floors, made by building service personnel,. Perhaps a spray can that injects an approved expanding fire resistant material into such openings after wires, pipes or ducts are run.

Exits

Stairways should be remote and separate from each other to avoid being put out of service by one incident, or being blocked by fire or smoke.

Escaping occupants had to reverse direction and go back up stairways in several instances due to dead end, corridor crossovers and locked exit doors from the stairwells. (See Locked Exits… in bibliography). "… the stairways were not continuous and at various points people had to transfer from one stairwell to another to continue there decent." (Hammett, July/Aug. p.35) The need for security is understood, but this suggests a lack of understanding of the function of, and critical need for, unobstructed availability and continuity of fire exit stairways in high rise buildings. Automatic fail-safe door latches should be installed and maintained throughout the stairways to unlock all exit-way doors in the event of fire. Whether ascending or descending, if a stairway suddenly fills with smoke, occupants should be able to exit these stairways immediately and find other stairs or areas of refuge, before they can be asphyxiated by smoke.

(Present city code allows stairway doors from the stair side to be locked except for 're-entry doors' every 4th floor. Once a person enters the stairway they cannot leave the stairshaft except by ascending or descending to the next 're-entry' floor.)

There seems to be a natural tendency for people to flee up the stairways, if the fire is below them; this may be a natural reaction, or possibly is used as a means to get to the roof. Fleeing up above a fire, has to be discouraged in fire drills and employee training, since it is a most dangerous practice, since smoke and heated gasses expand, produce pressure and become buoyant and rise up any available open shafts, including stairways. If stair enclosures are breached or self closing devices on doors are defective or inactivated, or standpipe hose lines are being used, smoke can enter and contaminate stairway exits. Carbon monoxide is a deadly toxic gas which causes disorientation, lethargy, stupor, unconsciousness and death. If persons are climbing the stairs and are not able to exit the stair shaft promptly when smoke enters, they may not make it out of the stair shaft. If fail safe unlocking devices cannot be fitted, the practice of using widely spaced, re-entry floor, doors needs to be discontinued.

Roof doors in stairways are required to be easily openable from the inside, in the City Codes, recognizing that some people may, in spite of the danger, attempt to make it to the roof. People may become trapped inside the stairwell by smoke, if roof doors are locked, as they were at the Towers.

Remotely controlled, hatches should be required over stair and other shafts to be activated from the fire command station for rapid safe ventilation of stairways and other shaft- ways to prevent smoke and heat buildup on the upper floors of high-rise buildings. (Van Holt) Stairways and other vertical shafts should be required to extend to the roof in order to provide this ventilation.

In order to increase exit capacity and assure availability, additional remote stairways could be installed at building outside corners or outside walls, in addition to those at the ends of the corridors and in the hardened core.

Stairways, corridors and other exit pathways, leading to, or between, rooms or stairways, or to the outside, should also be hardened to preserve their integrity and continuity; these both for occupant escape and fire department attack team safety. Sheetrock walls which can be destroyed by heavy water streams may still be advantageous for certain, room walls, off the corridors, to allow Fire Department water stream penetration during heavy stream attack or for easy forcible entry, for rescue or hose line advancement to flank a difficult fire.

Scissor stairs should be re-evaluated because of the possibility of both stairways being affected by a disruption of the enclosure. Also they violate the spirit of the 'remote' exit requirements, and add confusion in lettering stairways and confusion in locating alternate landing, standpipe outlet locations.

Elevators

Elevators frequently fail to provide adequate, safe Fire Department response to the upper floors of high-rise buildings. Provisions for fire, smoke and water resistive elevators, and impact protected, elevator shaft enclosures should be developed for Fire Dept. access to upper floors and handicapped rescue from upper floors. If we can ring an entire, 16-acre, foundation area with 3-foot thick reinforced concrete, 7 stories high, to keep the Hudson River out, we can ring the areas of stairs, elevators and lobbies on each floor with reinforced masonry, fire walls and provide pressurization and ventilation gaps to keep fire and smoke out. Fire proof, ventilated vestibules, as presently used in the 1938 code 'fire tower', smoke proof stairways, could be used. Pressurized 'areas of refuge' as elevator and stair landing areas, on each floor, could be used in conjunction with fire rated elevator shaft, doors. If the stair and elevator shafts, are not overly pressurized the excess air pressure should vent out of the exhaust vents near entrance doors, in these vestibules and not reach the fire to accelerate it. (In tests done by Chief O'Hagan stair pressurization was shown, at times, -when the door on the fire floor was opened to advance hose lines,- to accelerate the fires and spread smoke to remote areas of the building.) Door sills or some other means should be provided to keep water out of shafts. Elevator electrics, should be water, smoke and heat proof. Elevator escape, doors will have to be provided on every floor since breaching such hardened, shaft walls, for rescue of people in stalled elevators would be difficult.

Water Spray Extinguishing Systems- 'Sprinklers'

Full, water spray 'sprinkler' protection should be mandatory in all buildings, over 6 stories, or 75 feet in height, no matter what the building occupancy. We cannot always control the amount and type, of combustibles entering the buildings. Redundancy in water spray system risers, tanks, pumps and water supply mains, should be provided for high-rise buildings due to the numbers of people exposed to any fire incident, the increased Fire Dept, response time, the increased possibility of failure of any key element in the water supply, and pressure requirements for effective fire control. This redundancy should cover the entire chain, of water supply, to the heads, including street water mains, tanks (gravity, pressure and suction), pumps, power supply to pumps, valves, etc…

'Sprinkler' systems should be separate from Standpipe systems, since with 'combination systems' failure of piping or other element of the water supply chain, or heavy use of either system's supply, will affect the pressure and water supply, to the other system, possibly leaving areas deficient in, or devoid of, extinguishment capabilities. At fires where the sprinkler system is ineffective and possibly overloading floors the Fire Department might want to stop supplying the system while still using the standpipe hose lines. This may be impossible if the systems are interconnected.

(Since supplying water, from the street level, by standpipe, to the top floors of the world trade center would require pressures of over 600 psi, to overcome head and friction loss pressures, and sprinkler heads are only tested to 500 psi; I wonder if and how they took care of this pressure discrepancy when they took sprinklers off the standpipes.)

Increase water flow, pressure capacities, and protection of pipes, by increasing the size and strength, of pipes and hangers, and enclosing risers in hardened shafts. Qualified hydraulic engineers should be consulted to design water supply systems in high-rise buildings.

Presently areas between firewalls are commonly supplied by one riser, or main branch water line. Redundancy in supply could be provided by feeding ceiling grids, from two or more directions, by separate risers, or separate main branch lines, protected by check valves, at the grid connections to the risers, or branch line connections. If one riser breaks or is otherwise depressurized the check valves would close, due to the pressure from the other riser(s) and the area would remain protected. This configuration will also alleviate the effects of a blockage in the piping, since water will be flowing in from two directions. It would also reduce friction losses, increasing the supply capacity (Gallons per minute), assuring proper pressures at the heads, and reserve capacity in case of fire extension. It will also, most times, allow repairs to be done on the system without shutting it down entirely.

Separate risers should be supplied by separate water mains, separate tanks-(both gravity and pressure tanks), separate pumps with separate, or reserve, power supplies, and have separate OSY valves and check valves. This would double the dependability of water spray systems and provide extra capacity, for fires which spread through a fire wall, or fire barrier floor to involve another area. A large percentage of sprinkler failures result from inadvertently closed valves. Having separate risers will reduce this hazard greatly since one valve closure will not shut down the system.

It may be possible to supply alternate floors with different risers, by running two risers in each hardened shaft, or in separate shafts. In case of floor collapse, the spray heads on the floor above the collapsed floor would remain pressurized to control the fire, after the fire-floor, branch pipes were destroyed. Alternatively, automatic sealing devices for broken pipes could be developed.

Areas of Refuge- Fire/Blast Wall

In order to mitigate the deficiencies, in today's high-rise construction, which allow spread of heat and smoke, from floor to floor and the difficulty evacuating, large numbers of people, and in order to provide, rapidly accessible, safe areas, tall buildings should be divided into a minimum of two sections, from cellar to roof, by a four hour, reinforced, high-strength concrete, fire division wall (or blast wall) with no openings permitted, except horizontal exits, with double, self closing, automatic fire doors between the sections. Smoke detectors, associated with magnetic door closing devices, have proven effective and are easily tested for operation by, in house, fire safety personnel. Each side or section, of the building, should have smoke proof stairs and elevators available, for further egress and Fire Dept. access.

Separate heating, ventilating and air conditioning systems, plumbing, communications and electrical wiring, systems could be confined to each side (section or division) of the fire/blast wall. Before smoke can spread through the HVAC system, or other wire, pipe or duct openings; or fire burn through the floors, and the resulting heat and toxic gas buildup contaminate an entire side, (as happened on the floors above the fires at the Towers) people could escape through a horizontal exit, in the fire wall, to the other side and find immediate refuge, in the adjoining section, with ample time for escape to the street. The concern about high-rise buildings, not containing fires, from floor to floor, would be mitigated by allowing people, immediate refuge, through the horizontal exit doors to the adjoining section, in case of smoke buildup or overcrowded stairways in the affected section. Handicapped in wheelchairs or people with disabilities could easily be evacuated, through the horizontal exits and await further rescue if necessary, at the smoke proof elevator lobby, rather than slowing down emergency stair evacuation.

After the location of the fire is definitively determined, people could be instructed, to evacuate horizontally, to the correct adjoining section, by the building safety director, using loudspeakers. Safe response, for Fire Dept. personnel, could be had using elevators in the clear section, for rescue and fire control operations.

The Fire Dept. would have access, through the clear building section, to the fire floor and floors above. Proceeding from a safe staging area in the clear section, rescue, fire extinguishment or containment of the fire, could be more safely accomplished with a relatively safe area, immediately available, in the adjoining section.

This four hour, fire division wall configuration would allow any people, above the fire, or those who mistakenly fled up the stairs, above the fire, or to the roof, to be evacuated or rescued by being led, to the adjoining clear section, from the floors or the roof and than down the stairs. It would also allow safe, roof access, by Firefighters, traveling through the clear section to the roof, for ventilation, over the stair, elevator shafts and other shafts in the affected section(s), thus alleviating smoke, heat and combustible gas buildup, on the upper floors, allow controlled ventilation for line advancement, and also limit smoke spread to the clear section. All stairways should be required to extend to the roof in each section, for fire gas ventilation and rescue of people on the roof or stranded on the stairs.

It is extremely difficult to operate hose lines directly out of a stairway while it is exposed to smoke and heat conditions from a large area floor fire. If the line is advanced on to the floor the stream can draw heated gasses in behind the hose crew forcing retreat. Two 4hr. parallel, fire walls could be installed to form, hardened corridors, with smoke proof, stairways at each end, and possibly the middle, for firefighter attack line safety and to gain room for the men and position for hose lines. Extra standpipe outlets could be positioned near the horizontal exits, in the corridors.

Extra tall or large area buildings could be, additionally, divided into quarters, by a second 4 hr, fire division/ blast wall perpendicular to the first, to provide extra options for fire/blast confinement and 'area of refuge' safety, in fire or other emergency. In buildings, subject to the possibility of colliding planes, these blast walls could be robust; designed to contain the plane to one side or section leaving the other(s) free for evacuation. Two parallel fire/ blast walls could be used to form the corridor with horizontal exit doors, offset from each other to divert blast pressure waves, passing these walls. Each section, of the building, should be built to be structurally stable on its own, even if one section experiences partial collapse. These fire walls would also act as shear walls and add considerable lateral stability to the building.

Air Conditioning

Central air conditioning systems servicing many floors have shown to accelerate the fire and spread deadly smoke through out many floors in high-rise fires. Chief Dunn's recommendation that air conditioning systems should cover only one or two floors should be implemented.

Inspections

"Controlled inspections" by engineers, hired by the owner/builder, should not be wholly depended upon. Building Dept. re-inspections and spot checks should additionally be instituted, to assure maintenance of documentation and compliance with corrective action.

Building inspectors should be a uniformed force and be required to take an oath to protect the lives and property of the people of the City of New York. There should be an Enforcement Section with peace officer status.

After Words

I am sure there will be many additional recommendations for Building Code improvements, which will be gleaned, from the WTC catastrophe. History has shown, and this tragedy has proved, that a good Fire Prevention and Building Codes, knowledgeable people administering them, and strong enforcement capabilities are necessary to build and maintain safe buildings. Critical code sections should be protected from special interest changes. As a good example; the NY City code Sect.27-339 subparagraph © #3 allowed unsprinklered floor areas of 15,000 square feet (three times the fire area that firefighters can extinguish with interior hand held hose lines), provided that smoke detectors were installed. (Instead of sprinkler systems?).

. The World Trade Center's vulnerability to fire, as confirmed by the fire spread and mode of collapse, is partially the result of the building industry's competition for, real estate dominance and financial reward, affecting the building codes over the years. The Port Authority of New York, New Jersey using corporate and public bond financing and the governmental power of the two-state agency to sidestep the already weakened, city building code requirements effectively reduced the fire resistance and suppression capabilities and collapse resistance, in the Towers. The Government should disqualify itself from competing in the real estate industry and concentrate on regulating the competition between developers to assure fire safe building construction standards and the life safety of the people. The actual fire is the ultimate test of codes and construction practices and at the World Trade Center Towers, failed the test twice.

Bibliography; and Internet Links:

Bazant, Zdenek P., Why Did the World Trade Center Collapse ? - Simple Analysis, Journal of Engineering Mechanics ASCE, 9/13/01-9/28/01.

Brannigan, Francis L., - Building Construction for the Fire Service first edition 1971, second edition 1982, & third edition 1992, National Fire Protection Association – Batterymarch Park, Quincy, Massachusetts.

Dissecting the Collapses, Civil Engineering Magazine, May 2002, http://www.pubs.acse.org/ceonline/ceonline02/0502feat.html.

Clifton, G. Charles, "Collapse of the World Trade Center Towers", HERA structural engineer, HERA, Manukau City, New Zealand, http:// www hera.org.NZ/PDF Filer/World Trade Center. PDF

Clifton, G. Charles, "Elabortion…." 12/13/01, Ibid,

Clifton, G Charles, "The Collapse of the World Trade Center Towers -Causes and Lessons Learned" , HERA structural engineer Z:\Structural\Metals Industry Conference 2002\CC Paper World Trade Center\WTC paper for metals conference – v1.doc

Clifton, G. Charles, et.al.; Design of Multi-Story Steel Framed Buildings With Unprotected Secondary Beams or Joists for Dependable Inelastic Response in Severe Fires; Proceedings of the Second International Workshop on Structures in Fire, 2002 University of Canterbury, Christchurch, 2002, pp.151-716.

Darton, Eric, Divided We Stand, A Biography of New York's World Trade Center, Basic Books 1999, http://ericdarton.net/

Corley W.G., "Engineers Set the Record Straight on Trade Center Study Results", (11/04/2002), by Nadine M. Post, McGraw Hill Construction, ENR, REF = http://enr.construction.com/news/buildings/archives/021104a.asp

Dunn, Vincent –"Collapse of Burning Buildings, a guide to fireground safety", Fire Engineering Books and Videos, a Division of Penn Well Publishing Company, 1988

Dunn, V, "Why Can't the Fire Service Extinguish fires in High-rise Buildings", Fire Engineering Dec. 1995, Penn Well Corporation, 21-00 Route 208, South Fair Lawn, NJ, 07410-2602

Glanz, James and Lipton, Eric, New York Times Magazine, The Height of Ambition: Parts One through Seven, Sept. 8, 2002. http://www.nytimes.com/2002/09/08/magazine/08WTC1

Hamburger, Ronald. In "Structural engineer describes collapse of the World Trade Center towers" by Mark Shwartz -Stanford Report, Dec3, 2001

Hammett, Kingsley, Editor, 2405 Maclovia Lane, Santa Fe, NM 87505, www.designerbuildermagazine.com/

Malott, Jim, Why the World Trade Center Collapsed: an architect's assessment, Designer Builder Magazine, Kingsley

New Scientist.com, "Twin tower collapse theory challenged". Feb. 12, 2003, http://www.newscientist.com/news/news/jsp?id=ns99993354

NIST, "Progress Report on the Federal Building and Fire Safety Investigation of the World Trade Center Disaster", May 2003, NIST Special Publication 1000-3.

NIST- Analysis of Needs and Existing Capabilities for Full-Scale Fire Resistance Testing , Jesse Beitel, & Nestor Iwankiw, NIST GCR 02-843.

Norum, Harry C. "Backdraft" With New York Firemen (WNYF) 4th Issue 1980.
Gillespie, Angus Kress, "Twin Towers", Rutgers University Press, New Jersey, and London, 1999.

.O'Hagan, John T., "High Rise/Fire and Life Safety", Dun-Donnelley Pub. Corp. N.Y C., NY, 1977

Rapp, Robert E., "The World Trade Center:" An Architectural and Engineering Milestone, Contemporary Steel Design Vol.,1 No. 4

Quintiere, J.G. & M. di Marzo & R. Becker, A Suggested Cause of the Fire –Induced Collapse of the World Trade Towers, & update "Modification to the Thermal Analysis" September 2002, NFPA, Fire Safety Journal, Vol. 307 pp. 707

Usmani, A.S., Chung, Y.C. and Torero, J.L., School of Engineering & Electronics, University of Edinburgh, UK "How Did the WTC Towers Collapse: A New Theory", Unpublished Report; to be published in Fire Safety Journal, http://www.see.ed.ac.uk/

Usmani, A.S. , J.M. Rotter, S. Lamont, A.M. Sanad, and M. Gillie, Fundamental principles of structural behavior under thermal effects, "Fire Safety Journal, 36:721-744, 2001, http://www.civ.ed.ac.uk/research/fire/tecnicalreports.html.

World Trade Center, Locked Stairwells/ Evacuation Problems Accumulating Problems On… © 2001 Mike Barkly, http://www.mjbarkl.com/locked.htm

Walsh and Marks, "Firefighting Strategy and Leadership" Second Ed., McGraw Hill, 1976