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OPTIMIZATION OF TIED LATERAL SYSTEMS FOR TALL BUILDINGS DESIGN

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ABSTRACT

I sistemi di controvento rappresentano un aspetto molto importante nella progettazione di edifici alti e necessitano di essere presi in considerazione fin dalle prime fasi della progettazione, a partire dal conceptual design.

Il lavoro proposto si propone di combinare, all'interno della stessa struttura, un comportamento sufficientemente rigido in condizioni di servizio e un comportamento duttile localizzato sotto azioni sismiche elevate, creando quindi un *tied lateral system* dove efficienza strutturale e valenza architettonica contribuiscono a creare una struttura di grande valore. Un *tied lateral system* coniuga le proprietà di rigidezza di un sistema controventato ottimizzato con quelle duttili di un sistema di controventamento eccentrico tradizionale.

I principali aspetti e le caratteristiche dell'ottimizzazione topologica, applicata ad edifici alti, sono stati richiamati, assieme ai concetti base dei sistemi di controvento eccentrici. Un'analisi qualitativa è stata effettuata su un modello esemplificativo del caso studio successivamente analizzato: una *sizing optimization* è implementata, ottenendo la distribuzione di aree ottimali nel rispetto di uno spostamento laterale imposto, seguita poi da un'analisi plastica localizzata, realizzata attraverso la creazione di cerniere plastiche nei *link*. Infine sono stati ricavati dei criteri per modificare le rigidezze relative dei vari elementi, al fine di ottenere un comportamento duttile il più uniforme possibile.

Successivamente viene trattato un caso studio di edificio alto, applicando la metodologia appena ricavata. È stata utilizzata una condizione di carico da vento per il *sizing* degli elementi principali del sistema laterale, che viene effettuato attraverso un programma *in-house,* ed è invece poi stato definito un carico sismico fittizio per la definizione e il controllo del meccanismo plastico attivatosi nei *link*.

Concludendo, il comportamento duttile uniforme manifestatosi nella struttura sotto carico sismico, assieme alla rigidezza espressa sotto carico da vento, hanno validato la metodologia proposta, estendendone l'applicabilità al più generale mondo dei palazzi alti.

ABSTRACT

Lateral resisting systems are a very important issue in tall buildings design and they should be tackled in the very beginning of the design, in the conceptual design phase.

The following work focuses on the combination of both a stiff and a localized ductile behavior in the same structural system, thereby creating a tied lateral system where structural efficiency and architectural beauty can coexist, producing an extremely valuable structure. A tied lateral system encloses inside of its definitions the stiffness feature of an optimized lateral resisting frame, with the addition of the ductile behavior coming from an eccentrically braced frame system (EBF) configuration.

Main concepts and design criteria of topology optimization for high-rise buildings design and of EBF systems have been recalled in order to motivate a suitable frame layout. A sensitivity analysis is carried out on a simplified model representative of the case study subsequently analyzed. Sizing optimization is implemented in order to get the optimal area distribution in each structural element to satisfy stiffness criteria under lateral loads, expressed as target lateral displacement. After that, a plastic design is performed with the aim of creating localized plastic hinges in the link beams. A methodology is thus identified to modify each structural element category to obtain a ductile behavior as uniform as possible, along the whole height of the structure, while preserving the optimality of area distribution and a stiff structural behavior.

A case study of a high-rise building is then studied, in which the methodology described in the simple model has been applied. Two different loading conditions are implemented, one for the elastic sizing and the other, featuring larger loads, for

the activation of the ductile mechanism. The braced system is sized through an in-house software and a plastic analysis is performed on the link beams.

As a conclusion, the good and uniform ductile behavior exhibited by the real structure, combined with the high rigidity that the lateral optimized frame provides the entire structure with, prove the validity of the conceived methodology workflow. Its general validity extends its applicability to the world of tall buildings design.

RIASSUNTO DELLA TESI

I sistemi di controvento (*lateral braced frames*) sono uno degli aspetti strutturali più rilevanti nella progettazione di edifici alti. Lo scopo di questo lavoro è studiare una soluzione di controvento ibrida, esposta in facciata, che racchiuda la rigidezza propria di un layout ottimizzato con il comportamento duttile dato da elementi dissipativi, anche chiamati *fuses*. Servendosi della teoria dell'ottimizzazione strutturale è possibile, fin dalle prime fasi del progetto, arrivare a definire la miglior geometria per una data condizione di carico e di vincolo. La fase successiva consiste nell'inserire nel sistema una serie di *links*, travi a doppia T nel caso in esame, che, attraverso il loro comportamento plastico localizzato, conferiscano alla struttura un adeguato comportamento duttile. Grazie all'interazione di queste due tipologie strutturali è possibile sviluppare una struttura capace di soddisfare le sempre più crescenti performance richieste nella progettazione di edifici alti, sia in termini di efficienza strutturale che di valenza architettonica.

Si rende dunque inizialmente necessario studiare questi due elementi strutturali separatamente, evidenziandone il comportamento e studiandone pregi e difetti, per poi trarre il massimo vantaggio dalla loro interazione. Si vuole così giungere alla realizzazione di una struttura che soddisfi adeguati valori di rigidezza in condizioni di servizio e manifesti un adeguato comportamento duttile per condizioni eccezionali di carico sismico. Queste sono infatti le due condizioni di carico principali cui l'edificio, oggetto del *case study*, è soggetto, essendo ubicato in prossimità di una baia nel sud della China, zona ad elevata sismicità e interessata da forti venti.

A partire dagli ultimi decenni del diciannovesimo secolo, grazie alla sempre più incalzante sfida che si poneva nella progettazione di edifici alti, l'importanza di sistemi di controvento per far fronte a carichi laterali assunse sempre più importanza. La maggior parte di queste strutture era fino a quel periodo dotata di telai in acciaio con diagonali facenti riferimento a diverse configurazioni come K, X ed eccentriche: le diagonali erano solitamente collocate all'interno del *core*, che era spesso collocato all'interno dell'edificio.

A partire da questa concezione, intorno al 1960, si ebbe un primo sviluppo con l'introduzione delle strutture così dette *tube in tube*. Da questo momento i progettisti capirono l'importanza e il bisogno di un sistema duale in grado di sostenere i carichi di gravità con un *core* centrale in calcestruzzo armato e di resistere i carichi laterali attraverso un telaio in acciaio posto nel perimetro esterno.

John Hancock Center (sinistra) e Sears Tower (destra), Chicago, IL.

Uno dei migliori esempi che testimonia questo sviluppo, attraverso il sistema laterale *diagonal tapered trusses*, risulta essere il John Hancock Center a Chigago, progettato da Bruce Graham e Fazlur R. Khan, di Skidmore, Owings & Merrill LLP, e ultimato nel 1969. La sua caratteristica principale è la struttura diagonale a "X" in facciata. Essa è presente su ogni lato per massimizzarne l'efficienza strutturale ed è connessa in modo tale da trasferire il carico alle colonne e vice versa. Delle travi sono presenti alle quote in cui le diagonali intersecano le colonne in modo da scaricare i carichi verticali distribuiti sulle colonne. La struttura in acciaio, esposta in facciata, si presenta snella e leggera e allo stesso tempo trasmette un senso di robustezza e stabilità.

In questa fase storica le strutture, come quella appena citata, nacquero e vennero sviluppate a partire da concetti strutturali che andavano via via consolidandosi, fino ad essere chiaramente espresse nell'architettura dell'edificio.

La Sears Tower a Chicago è un altro esempio dell'utilizzo del sistema strutturale *tube in tube* per il progetto di un edifico alto. In questo caso la struttura è dotata di nove strutture tubolari, con outriggers e travi reticolari a diversi livelli, essenziali per le prestazioni strutturali dell'edificio. Le strutture tubolari come quelle appena citate sono state sviluppate nel corso degli anni, permettendo la nascita di nuovi sistemi strutturali chiamati *diagrid systems.*

Nei *diagrid systems* quasi tutte le colonne convenzionali sono eliminate. Ciò è possibile poiché gli elementi diagonali sono in grado, grazie alla loro configurazione triangolare, di sostenere sia carichi laterali, sia carichi di gravità, mentre le diagonali di sistemi controventati convenzionali possono sostenere solo i carichi laterali. Confrontate con le strutture tubolari senza diagonali, i *diagrid systems* sono molto più efficaci nel minimizzare la deformazione in quanto possono resistere all'azione tagliante attraverso la resistenza assiale degli elementi diagonali, mentre le strutture tubolari resistono al taglio grazie alla flessione delle colonne verticali.

Nella Bank of China Tower, costruita nel 1990 a Hong Kong, appare chiaramente il ruolo strutturale delle diagonali esposte in facciata. Inoltre la rigidezza propria delle *diagrid systems* è estremamente utile non solo per carichi statici, ma anche per carichi dinamici.

Appurato il grande beneficio strutturale conferito a questa tipologia di strutture nell'utilizzare geometrie triangolari, un commento va speso rispetto alle connessioni richieste per collegare diagonali e colonne. La loro realizzazione, visto l'alto livello di dettaglio richiesto, è un aspetto che va preso in considerazione fin dalle prime fasi del progetto: la prefabbricazione in stabilimento si rende indispensabile per ridurre i costi di realizzazione all'interno di una struttura che tende ad essere, già di per sé, più costosa di altre strutture convenzionali.

Negli ultimi anni la caratteristica principale delle strutture *diagrid* si è dimostrata essere la possibilità di coniugare allo stesso tempo efficienza strutturale e valenza architettonica. Se da un lato l'importanza strutturale delle diagonali era già consolidata, d'altro canto il loro potenziale estetico non fu fin dall'inizio considerato e valorizzato. La progettazione di queste strutture si rifaceva tipicamente a geometrie triangolari di *45* o *60* gradi. Molti studi sono successivamente stati condotti nell'identificare le configurazioni migliori, in termini di angoli tra le diagonali, e i parametri che ne caratterizzano l'efficacia strutturale. Usando ad esempio metodologie come l'ottimizzazione topologica, è difatti possibile arrivare a definire la geometria ottimale per sfruttare al massimo le capacità strutturali e al contempo utilizzare un quantitativo minore di materiale.

Questa nuova metodologia nella progettazione di edifici alti ha dato la possibilità di conciliare nella stessa struttura sia aspetti strutturali che valenza architettonica ed ha portato alla creazione di edifici efficienti ed al contempo eleganti e unici.

Un altro importante aspetto da considerare riguarda il fatto che, nella progettazione di sistemi resistenti laterali di edifici alti attraverso l'ottimizzazione topologica, si considera che la struttura lavori in campo elastico. Ciò è possibile dove la condizione di carico predominante è quella da vento mentre quella sismica non è di particolare rilevanza.

La situazione è completamente differente quando si tratta della progettazione di edifici alti in zona sismica. Soprattutto negli ultimi decenni, molte strutture, nell'ordine dei 300-400 metri e superiori, sono state proposte in zone come China, California e Giappone, dove la condizione di carico sismico è la condizione di carico predominante da considerare nell'approccio alla progettazione. Nel progettare strutture dagli elevati requisiti, in termini di prestazioni richieste, soggette ad azioni sismiche di elevate intensità, si rende necessario inserire sistemi dissipativi che permettano loro di resistere ad eventi sismici eccezionali, evitando il collasso. Per far ciò, se si dovesse fare affidamento alla sola progettazione in campo elastico, si arriverebbe alla definizione di sistemi di controvento laterali di dimensioni notevoli, costosi e difficilmente realizzabili che produrrebbero cattive performance in termini di comfort. Si rende quindi necessario dotare il sistema strutturale di elementi dissipativi.

Il meccanismo plastico di dissipazione può essere ottenuto tramite l'inserimento, all'interno della geometria del sistema resistente laterale, di particolari dispositivi dissipativi chiamati *fuses*. Il conetto di *fuses* consiste nel progettare degli elementi "sacrificali" in cui concentrare le deformazioni plastiche e dissipare quindi energia, preservando l'integrità degli elementi circostanti che continuano a rimanere in campo elastico. Questa strategia è già stata utilizzata in altre applicazioni nei campi strutturale e industriale, con la peculiarità che l'elemento deputato alla dissipazione, una volta sviluppato il comportamento duttile, può essere sostituito con un nuovo elemento senza compromettere la funzionalità della struttura in cui è inserito.

Questo comportamento, in cui elementi a comportamento elastico e plastico agiscono assieme nella stessa struttura, è presente nella più semplice tipologia strutturale dei sistemi di controvento eccentrici (*eccentrically braced frames*): questi sistemi forniscono sia rigidezza sia un comportamento duttile grazie a un elemento corto di trave, spesso a doppia T, detto *link beam*. Molti studi sono stati condotti negli ultimi 40 anni sui sistemi di controvento eccentrici e sulle loro proprietà, in particolare migliorando il comportamento duttile attraverso la modifica delle caratteristiche sezionali del *link*.

Un sistema resistente applicato alla progettazione di edifici alti che fornisca contemporaneamente rigidezza e comportamento duttile, combinando le proprietà di sistemi di controvento eccentrici con i risultati della ottimizzazione topologica, può essere definito come *tied lateral system*.

Nel Capitolo 2 si affrontano le procedure di ottimizzazione. In generale l'ottimizzazione strutturale racchiude al suo interno un gran numero di metodologie, quali le ottimizzazioni per forma e per dimensione oppure procedure simili ad algoritmi genetici. La base comune di questi metodi è la ricerca della miglior soluzione che soddisfi un requisito o una prestazione richiesta. È quindi possibile, a seconda delle necessità del progettista, scegliere accuratamente il tipo di ottimizzazione da svolgere prendendo in considerazione le varie proprietà che la caratterizzano. Nel secondo capitolo si è affrontato il tema dell'ottimizzazione strutturale approfondendo da un lato la *topology optimization* (ottimizzazione topologica) e dall'altro la *size optimization* (ottimizzazione dimensionale).

Per quanto riguarda l'ottimizzazione topologica, essa consiste nel determinare la giusta posizione dove concentrare il materiale all'interno di un dominio definito per uno specifico obiettivo, basandosi sui carichi applicati, le condizioni di vincolo e il volume della struttura finale. Tali obiettivi possono essere uno spostamento limite o un valore di spostamento prestabilito, la minimizzazione del lavoro interno etc. Il dominio definito per svolgere l'ottimizzazione può essere soggetto ad altri numerosi vincoli, quali possono essere la posizione e la dimensione di aperture all'interno della struttura, la forma e la geometria di alcuni elementi od il tipo di connessione tra di essi. La soluzione finale, nel rispetto di questi vincoli, può avere una qualsiasi forma o dimensione ed è ottenuta suddividendo il domino in una serie di piccoli elementi, secondo la modalità propria del metodo degli elementi finiti. Ognuno di questi elementi è caratterizzato da un valore di densità che può essere nullo, oppure unitario, a seconda dell'importanza o meno nel dominio dell'elemento stesso. Il risultato è un dominio in cui emerge in fasi sempre più definite la configurazione finale della miglior struttura atta a soddisfare il requisito imposto.

La configurazione finale che emerge della struttura è strumento di lavoro sia per gli ingegneri che per gli architetti. Per gli ingegneri l'ottimizzazione topologica mostra quali sono i percorsi di carico previlegiati o gli elementi essenziali ed ineliminabili della struttura che hanno portato al soddisfacimento della prestazione richiesta. Per gli architetti invece la nuova forma, che nasce come una configurazione che risponde a criteri puramente strutturali, è luogo di studio per sviluppare nuovi *concepts*. La validità dell'ottimizzazione topologica risiede proprio nel fatto che combina l'efficienza strutturale con una visione architettonica nuova dell'elemento resistente. Dando uno sguardo alla recente storia architettonica basta guardare a quegli edifici che negli ultimi decenni ne hanno cambiato il corso per convincersi della necessità di una cooperazione tra valore architettonico e sensibilità strutturale.

La *size optimization* è invece uno strumento più specifico che fornisce la distribuzione migliore di area sezionale tra gli elementi della struttura, sempre al fine del raggiungimento di un obiettivo predefinito. Nell'utilizzo di questo metodo la forma o le connessioni tra i vari elementi sono fissate e gli elementi stessi non possono essere rimossi all'interno del processo. Per applicare la *size optimization* al caso di studio, che viene presentato nell'elaborato, si è fatto uso dell'*Energy based design for lateral systems*, un metodo di *sizing* energetico proposto da W.F. Baker in un suo articolo nel 1992. Tale metodo di ottimizzazione viene dimostrato matematicamente rifacendosi alla trattazione originale di Baker e successivamente vengono proposte alcune applicazioni per approfondire alcuni argomenti, come l'uso di tale metodo in strutture iperstatiche con procedura iterativa.

Grazie alla *size optimization* si ottiene una distribuzione di area che è inversamente proporzionale allo spostamento imposto come obiettivo per l'ottimizzazione. Ciò significa che, a parità di carichi applicati, volendo solamente cambiare lo spostamento massimo assegnato, basterà scalare adeguatamente anche tutti i valori di area di ogni singolo elemento. Un aspetto di problematicità è invece l'assegnazione dell'inerzia della sezione una volta noto il valore dell'area ottimale. Difatti, anche gli elementi che lavorano principalmente sotto uno stato di sforzo flessionale ricevono come output dell'ottimizzazione un valore di area che deve quindi essere tradotto nel corrispondente valore di inerzia. Viene dunque linearizzata la relazione area-inerzia che, se da un lato consente di superare agilmente questo problema, dall'altro richiede che la legge lineare usata non venga cambiata una volta scelte le sezioni ottimizzate.

I concetti appresi grazie alle due applicazioni discusse nell'elaborato danno importanti indicazioni anche rispetto all'uso del software di ottimizzazione utilizzato per il dimensionamento della struttura del caso di studio.

L’approccio seguito nella normale progettazione, facendo uso di strumenti di ottimizzazione, viene infine chiarificato per mezzo di uno schema che ne illustra le principali fasi. Prima di tutto si dimensionano gli elementi verticali facendo rifermento alle condizioni di carico per gravità, quindi si applica l’ottimizzazione per carichi laterali. Viene di conseguenza identificata la configurazione ottimale per la trasmissione del carico e creato il modello del telaio resistente. La *size optimization* è applicata ai singoli elementi, se necessario si procede all’iterazione ed infine si ottiene la struttura finale ottimizzata. Tale schema logico è puramente indicativo del processo da seguire ed a seconda delle necessità di ingegneri e architetti può essere modificato adeguatamente.

Nel Capitolo 3 vengono presentati i sistemi di controvento eccentrici, *eccentrically braced frames – EBFs*, che sono sistemi resistenti laterali per strutture in acciaio, considerati una soluzione ibrida che nasce dall’unione tra un telaio a momento resistente e un sistema di controvento concentrico. La peculiarità degli EBFs, rispetto alle strutture sismo-resistenti, è la loro alta rigidezza combinata con una elevata duttilità e capacità di dissipazione plastica. Questa caratteristica, che contraddistingue gli EBFs, risiede nella presenza di una trave corta, detta *link*, a cui almeno una estremità di ogni elemento diagonale è connessa. La trasmissione delle forze tra gli elementi avviene in modo tale che sul *link* siano concentrati sforzi di taglio e di flessione, mentre la restante parte del telaio rimane soggetta prevalentemente ad azione assiale, sebbene per equilibrio vi siano presenti anche taglio e flessione. La lunghezza del *link* definisce l’eccentricità dell’EBF e ne caratterizza di conseguenza il comportamento, essa viene indicata con *e*.

La duttilità degli EBF è attribuibile a due fattori. Prima di tutto il comportamento inelastico sotto carichi di elevata entità è concentrata esclusivamente nei *link*, che quindi sono appositamente progettati per sostenere grandi deformazioni in campo plastico senza perdere di resistenza. In secondo luogo la restante parte del telaio è progettata in modo tale da evitare fenomeni di instabilità e mantenere un’adeguata resistenza sotto i carichi agenti. Lo snervamento dei *link*, che può essere accuratamente valutato tramite test di laboratorio, permette di limitare la massima forza trasferita agli altri elementi, agendo quindi come elemento sacrificale nei confronti delle forze esterne che agiscono su tutti gli elementi dell'EBF. Se vengono osservate queste due regole di progettazione, rispetto al *link* ed alla restante parte del telaio citate prima, si permette un comportamento isteretico stabile anche sotto le condizioni di carico più stringenti.

Le proprietà fino a qui citate dipendono strettamente dalla lunghezza *e* del *link*: se da un lato più il *link* è corto più la rigidezza aumenta avvicinandosi al comportamento di un sistema di controventamento concentrico, dall'altro *link* troppo piccoli sarebbero sede di deformazioni inelastiche eccessive. La lunghezza del *link* determina anche il tipo di comportamento plastico, facendo da discrimine tra la formazione di cerniere plastiche a momento o a taglio. *Link* "corti" saranno soggetti a snervamento per sforzi di taglio mentre quelli "lunghi" per sforzi di flessione; sperimentalmente si evidenzia come i primi danno luogo a un comportamento migliore degli ultimi. Nel caso di studio si è dunque cercato di avere tutti *link* che si plasticizzassero sotto sforzi di taglio.

I test per valutare la entità di dissipazione e la capacità plastica sono legati alla stima sperimentale della rotazione del *link*. Tale rotazione è definibile teoricamente tramite l'analisi del meccanismo plastico dell'EBF e viene poi verificata da test eseguiti su provini in laboratorio, dove si cerca di ricreare le condizioni reali di stato di sforzo a cui i *link* sono soggetti. Riguardo a questi test, una proprietà importante è definita dalla presenza o meno degli irrigidimenti d'anima, che migliorano sensibilmente le performance in campo inelastico rispetto ai parametri di dissipazione e duttilità.

Prendendo in considerazione il caso di snervamento per sforzi di taglio, la rotazione plastica può essere correttamente stimata dallo spostamento relativo dell'estremo del *link* diviso per la sua lunghezza; la componente elastica, essendo molto piccola, può essere trascurata nel calcolo. I provini con irrigidimenti raggiungono sperimentalmente valori di rotazione plastica di circa ±*0.10 rad*. Si può anche verificare sperimentalmente che la resistenza ultima dei provini dei *link* raggiunge, con anime irrigidite, *1.9* volte la resistenza a snervamento nominale. Lo stato di sforzo significativo a cui il *link* è sottoposto e la grande capacità di rotazione che gli viene richiesta richiedono che la progettazione di tale elemento sia fatta nel modo più accurato possibile, per evitare rotture improvvise o crisi che porterebbero ad un immediato collasso. I dati sperimentali mostrati nell'elaborato e le considerazioni riguardo al comportamento plastico sono relative al comportamento di *link* con snervamento per taglio e, se applicate ad altre tipologie, potrebbero non essere più appropriate.

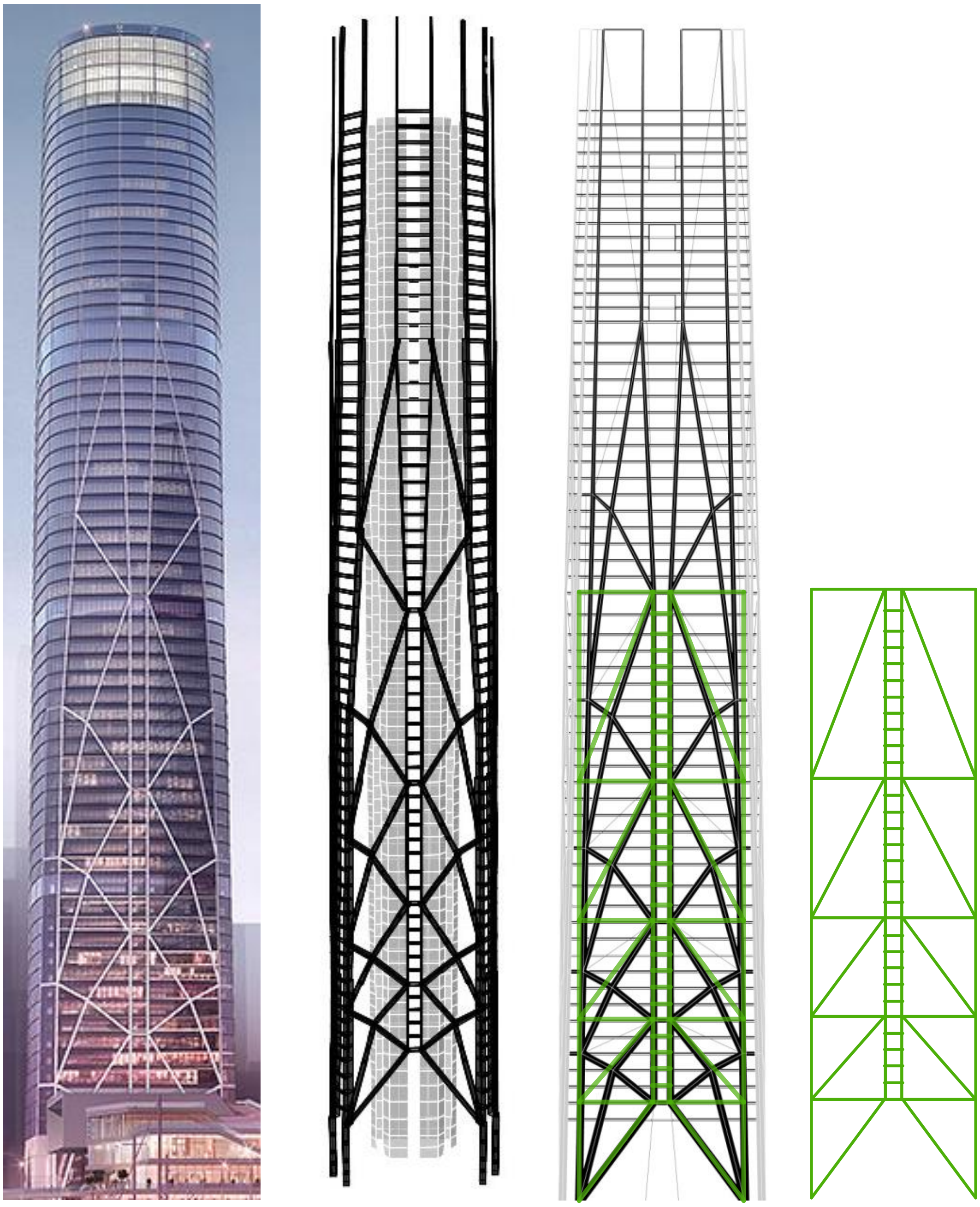
Sin dalle prima fasi della progettazione è anche necessario stabilire la configurazione degli elementi. Generalmente una linea guida nello scegliere tale configurazione è di evitare inclinazioni della diagonale rispetto all'orizzontale più piccole di *40* gradi poiché, tanto più piccolo è questo angolo, tanto più elevata è la forza assiale che impegna gli elementi del telaio dando luogo a potenziali instabilità. Il progetto del *link* viene fatto per primo e gli altri elementi vengono dimensionati di conseguenza. Con questo approccio il *link* risulta dimensionato secondo normativa per resistere alla forza di progetto mentre gli altri elementi sono progettati per resistere alle forze generate dal completo snervamento e successivo incrudimento del *link.* Tale forza rappresenta il livello massimo di sollecitazione a cui il telaio può arrivare nella condizione di carico più stringente, quale potrebbe essere quella del sisma di progetto. Va anche ricordato che gli elementi del telaio, ad esclusione del *link*, sono dimensionati per rimanere in campo elastico, mentre il *link* deve essere l'unico elemento soggetto ad un comportamento non lineare.

A conclusione di questo argomento si sottolinea come sia sempre necessario valutare nello specifico le condizioni geometriche e di carico a cui sono sottoposti gli elementi per poter stabilire il comportamento più efficace che il sistema può esibire.

Dopo aver analizzato nello specifico le proprietà degli EBF e introdotto i concetti relativi alla *size optimization*, nel Capitolo 4 si propone lo studio di una semplice struttura, creata apposta per riprodurre il comportamento reale di un sistema laterale analizzato successivamente. Siccome l'obiettivo è quello di combinare l'efficienza strutturale offerta dagli EBFs con i vantaggi legati all'ottimizzazione, il comportamento non lineare deve essere tenuto in considerazione. La principale problematicità sta nel fatto che una delle ipotesi fondamentali per l'ottimizzazione è che le proprietà sezionali e del materiale siano costanti e dunque sarebbe impossibile da estendere al campo non lineare. Per superare questo problema la procedura iterativa sarebbe sicuramente un valido strumento, ma, ad ogni modo, effettuare un'ottimizzazione strutturale considerando un comportamento non lineare è complicato e richiederebbe lo sviluppo di un software specifico. È dunque necessario separare gli elementi che sono soggetti ad un comportamento plastico da quelli che, rimanendo in campo elastico, possono essere ottimizzati.

Il modello semplificato del sistema laterale è ottenuto combinando più volte, in altezza, un singolo EBF inserendo una serie di *link* nella zona centrale, collegandoli con due colonne al telaio resistente. In questo modo la complessità del sistema laterale reale del caso di studio è notevolmente semplificata, ma vengono mantenute le caratteristiche principali. Alla struttura sono applicate forze simmetriche puntuali che traducono l'effetto dei carichi laterali quali il vento e il sisma: tali valori non sono realistici ma vengono definiti solamente per poter analizzare in modo qualitativo il comportamento strutturale.

Prima di tutto viene eseguita una prima ottimizzazione per gli elementi del telaio e successivamente, grazie all'aiuto di un software strutturale (*SAP2000*), si ottengono i valori delle azioni taglianti che interessano i *link*. I valori di area ottimali sono ottenuti attraverso un foglio di calcolo *Excel* in cui è implementata la formula completa dell'*energy based design method* proposta da W.F. Baker. Sia dall'analisi strutturale che dal processo di ottimizzazione è possibile capire qual è il meccanismo principale che governa la struttura ed identificare come la distribuzione delle forze varia a seconda delle proprietà di rigidezza dei singoli elementi.



Struttura semplificata dell'edificio alto oggetto del caso di studio, gentile concessione di Skidmore, Owings & Merrill LLP (SOM)

La serie centrale di *link* può essere suddivisa in due categorie: i *link* principali, che sono quelli soggetti a più alti valori di taglio e momento, sono quelli a cui sono attaccate direttamente le diagonali e le travi, mentre i *link* secondari sono quelli che, collegati dalle colonne centrali, stanno tra un modulo EBF e il successivo.

La trasmissione delle forze avviene grazie al nodo in cui sono interconnesse la diagonale, il *link* principale, la trave e la colonna centrale. L'azione di taglio uscente dal *link* principale deve andare a sollecitare quelli secondari attraverso le due colonne centrali, che risultano dunque fondamentali nel coinvolgimento dei *link* secondari nel meccanismo dissipativo. Il risultato dell'ottimizzazione, se condotto in modo iterativo, porta ad ottenere un sistema in cui solamente il *link* principale è sollecitato, mentre sia le colonne che i *link* secondari risultano pressoché scarichi. Dall'analisi di tali fattori risulta conveniente suddividere la struttura in due parti: una parte responsabile della rigidezza del sistema che si mantiene in campo elastico e viene quindi ottimizzata, una seconda parte in cui invece si concentra il meccanismo di dissipazione, che viene dimensionato in modo tale da garantire un comportamento plastico il più omogeneo possibile.

Riguardo al comportamento plastico a cui sono soggetti i *link*, si vogliono ottenere proprietà simili a quelle già studiate nella sezione dedicata agli EBF e dunque li si dimensiona in modo che lo snervamento avvenga per sforzi di taglio. Tale obiettivo implica il vantaggio di dover definire una sola cerniera plastica per ogni *link*, essendo lo stato di sforzo di taglio costante sul *link*, ma anche di scegliere sezioni il cui momento plastico resistente si abbastanza più grande del taglio plastico. La cerniera plastica è definita con una legge elastica perfettamente plastica ed i parametri sono definiti attraverso le relative impostazioni presenti in *SAP2000*. Dopo aver inserito le cerniere plastiche nei *link*, sia principale che secondari, si procede ad una analisi pushover in cui si può osservare il comportamento dell'intera struttura sotto i carichi applicati.

Si sceglie di studiare il comportamento plastico attraverso una analisi pushover poiché tale metodo permette una valutazione semplice e immediata dell'evoluzione del meccanismo plastico. Alla struttura vengono quindi imposti piccoli incrementi di carico ed è così possibile sapere il relativo stato di sforzo e quello nelle cerniere plastiche. Alla fine dell'analisi pushover è possibile valutare se le cerniere plastiche si sono attivate tutte e in quale sequenza e quindi avere informazioni sull'uniformità o meno del meccanismo dissipativo.

È importante, in questa fase, poter stabilire le proprietà di rigidezza relativa che gli elementi del sistema laterale devono avere per permettere il miglior comportamento plastico dei *link*. Due sono gli aspetti primari da tenere in considerazione. Il primo riguarda la rigidezza delle colonne contrali: attraverso di esse è trasmessa l'azione di taglio che va a sollecitare i *link* secondari e dunque devono possedere una adeguata rigidezza assiale per trasmettere questa forza. Al contempo la rigidezza flessionale, sempre delle colonne centrali, deve essere tale da non dare luogo ad eccessive deformazioni a causa del momento associato al taglio presente nei *link*. Viene mostrato dunque, nell'elaborato, che colonne centrali troppo poco rigide non riescono a coinvolgere i *link* secondari ed allo stesso tempo sono sede di grandi deformazioni flessionali dovute al momento uscente dai *link*.

Il secondo aspetto riguarda la rigidezza relativa tra le colonne centrali e le diagonali. Colonne centrali troppo rigide, sia assialmente che flessionalmente, andrebbero a sostituire all'interno del sistema di controvento il ruolo giocato dalle diagonali stesse, alterando il funzionamento dell'EBF. Il sistema di diagonali deve poter essere l'elemento principale del sistema resistente, assorbendo le forze provenienti dai carichi laterali evitando che siano scaricate sulle colonne. Se le colonne sono quindi adeguatamente rigide rispetto ai *link* secondari, ma al contempo meno rigide delle diagonali, si può notare che quando il *link* principale si plasticizza il valore di momento nelle diagonali e nelle travi risulta costante. In altri termini se il *link* è già in fase di plasticizzazione, all'aumentare del carico l'azione di taglio associata è assorbita dalla colonna, che la distribuisce ai *link* secondari, e non va ad aumentare lo stato tensionale a flessione degli altri elementi che risultano perciò "protetti".

Queste proprietà, mostrate dagli elementi componenti il sistema resistente laterale, sono da tenere in considerazione nel dimensionamento al fine di raggiungere un buon livello di plasticizzazione della struttura. Esse verranno discusse e valutate anche nell'esempio di studio affrontato nel quinto capitolo.

Nella parte finale dell'elaborato, il Capitolo 5, si affronta lo studio del *tied lateral system* di un edificio alto *300 m,* che verrà realizzato in una zona costiera nel sud della Cina. Il *tied lateral system* risulta essere l'unione di due strutture differenti: una resistente ai carichi laterali, responsabile della rigidezza complessiva dell'edificio, e l'altra dove si concentra la dissipazione.

Rendering del case study (sinistra), gentile concessione di Skidmore, Owings & Merrill LLP (SOM)

La struttura resistente ai carichi laterali è il risultato di un processo di ottimizzazione topologica sviluppato dallo studio di Skidmore, Owings & Merrill LLP (SOM), in cui poi sono stati inseriti una serie di *link* lungo tutta la zona centrale per permettere alla struttura, sotto carico di sisma, di dissipare energia tramite la loro deformazione in campo inelastico. Dopo aver brevemente illustrato il progetto e la fase di ideazione della struttura laterale resistente, il primo passo svolto è quello di utilizzare un software in-house, chiamato *OPTimizer*, in cui è implementata la formula della *size optimization*, già utilizzata nelle applicazioni proposte precedentemente.

L'OPTimizer è un software appartenente ad una famiglia di applicazioni per l'ottimizzazione che è stato creato e sviluppato dallo studio di SOM. Lo scopo di questo programma è di automatizzare il processo di *size optimization* dando all'utente la possibilità di gestire in modo semplice una grande quantità di dati provenienti da strutture molto complesse, quali possono essere quelle degli edifici alti. Il software è in grado di ottimizzare la struttura rispetto a più combinazioni di carico differenti e mostrare una soluzione che soddisfi il target di spostamento richiesto. È anche possibile vincolare delle sezioni a non superare definiti valori di area, come nel caso in esame viene fatto per il core e per gli elementi del sistema dissipativo.

Come fatto per l'esempio del Capitolo 4, una volta eseguita l'ottimizzazione della struttura elastica, è possibile risalire allo stato di sforzo che sollecita gli elementi interessati dal comportamento non lineare e quindi procedere al loro dimensionamento. Il sistema di colonne centrali e *link* viene progettato per esibire un comportamento plastico che sia il più omogeneo possibile, modificando i valori di rigidezza de vari componenti. Uno dei problemi principali è quello della definizione dei *link*, lunghi *4 m*, in modo tale che si plasticizzino per effetto di sforzi di taglio. Tale requisito porta ad avere anime molto snelle per aumentare il rapporto tra momento plastico e taglio plastico e quindi necessitano di particolari attenzioni nella fase di progetto rispetto alle potenziali instabilità. Anche in questo caso, come in precedenza, lo studio è qualitativo e quindi i valori di carico sono indicativi di una situazione estrema che permetta di porre in risalto le principali caratteristiche della struttura.

Tramite l’analisi pushover si studia il meccanismo plastico, il quale prevede la formazione delle cerniere plastiche prima sui *link* principali e poi su quelli secondari, che riescono ad essere coinvolti tutti nel meccanismo dissipativo. Lo studio e il confronto di due diversi casi, uno in cui il comportamento plastico non è omogeneo e i *link* non sono tutti coinvolti, e un altro in cui il coinvolgimento è completo, permette di riscontrare le proprietà mostrate analizzando la struttura semplificata del Capitolo 4. Si può giungere dunque alla proposta di uno schema, riassunto di seguito, da seguire nel processo di progetto di una struttura laterale di un edificio alto in cui si voglia al contempo utilizzare i concetti propri dell’ottimizzazione strutturale e dotare la struttura di un comportamento duttile.

Il primo passo da fare è dimensionare per i carichi di gravità gli elementi della struttura, per poi evidenziare nella struttura le due componenti principali: il sistema resistente e il sistema dissipativo. Il sistema resistente viene ottimizzato in campo elastico per i carichi laterali, mentre nel sistema dissipativo, formato da colonne centrali e *link*, si dimensionano gli elementi affinché si verifichi, sotto i carichi, previsti la plasticizzazione. Questa operazione è sicuramente resa più semplice se la struttura, come nel caso di quella in esame, è già stata pensata per racchiudere queste due componenti di resistenza e duttilità. Nel caso in cui questa metodologia si voglia applicare a strutture laterali generiche, l’operazione di separazione dei due sistemi potrebbe risultare più complicata.

Dopo aver combinato i due sistemi e selezionato il meccanismo plastico, si deve verificare lo spostamento massimo della struttura. Da un lato l’ottimizzazione assicura il raggiungimento del target di spostamento che è stato inizialmente imposto ma, avendo cambiato le sezioni del sistema dissipativo per renderlo omogeneo e far avvenire il meccanismo plastico, sono di conseguenza cambiate anche le forze agenti in ogni elemento, e il nuovo valore di spostamento potrebbe essere più alto di quello voluto. È necessaria quindi una procedura iterativa in cui la struttura elastica venga ottimizzata nuovamente e le sezioni trovate per il sistema dissipativo vengano vincolate al loro valore, senza essere cambiate dal processo di ottimizzazione.

Quando il target di spostamento imposto viene raggiunto, con o senza iterazioni, bisogna verificare tutti gli elementi dal punto di vista della resistenza. Tale verifica solitamente è soddisfatta, soprattutto per strutture come gli edifici alti dove i parametri più stringenti sono quelli legati alla deformabilità. Nel caso comunque che qualche sezione non fosse verificata è possibile cambiare la sezione dell'elemento e ripetere la procedura iterativa con l'area vincolata a mantenere quel valore minimo. Alla fine di queste operazioni si ottiene la struttura finale ottimizzata in campo elastico in cui, sotto una determinata combinazione di carico, vengono attivati i meccanismi di dissipazione grazie alla plasticizzazione degli elementi appositamente progettati.

Il *tied lateral system* presentato nel caso studio si è dimostrato essere un sistema efficiente, capace di coniugare comportamento elastico e duttile sotto condizioni di carico diverse, ottenendo una nuova e innovativa tipologia strutturale per la progettazione di edifici alti. Il *workflow* presentato attraverso uno studio qualitativo su un modello semplificato, è stato confermato nell'applicazione al caso reale, dimostrandone l'applicabilità nell'ambito più generale degli edifici alti.

Le diagonali e le colonne esterne sono gli elementi che contribuiscono principalmente alla resistenza delle forze laterali, fornendo alla struttura la rigidezza necessaria per manifestare un buon comportamento, sia in condizioni di servizio che di sisma, e possono essere analizzate attraverso l'ottimizzazione strutturale in campo elastico. I *link* sono invece finalizzati alla dissipazione di energia in caso di sisma raro e devono essere dettagliati in modo che il meccanismo plastico sia il più uniforme possibile. Riguardo a ciò è molto importante gestire e controllare in modo corretto le rigidezze dei vari elementi, con particolare attenzione alle colonne centrali, le quali hanno un ruolo determinante nel coinvolgere tutti i *link*.

Un primo commento può essere fatto rispetto alla progettazione delle colonne del sistema in facciata, le quali possono essere divise in due gruppi: colonne esterne e colonne centrali. Un design tradizionale, basato sui carichi di gravità che la colonna deve sostenere sotto forma di azione assiale, è il caso delle colonne esterne, che trasferiscono il peso dell'intera struttura alle fondamenta. Invece la funzione principale delle colonne centrali è quella di coinvolgere i vari *link*, quindi la loro progettazione è primariamente volta all'attivazione di un comportamento plastico uniforme lungo lo sviluppo verticale della struttura. Anche l'inclinazione delle diagonali che collegano le colonne centrali con i vincoli di base è un fattore da tenere in considerazione e che, se cambiato, produrrebbe effetti su tutto il sistema laterale.

Una seconda osservazione riguarda la fattibilità realizzativa dei *link* e delle connessioni tra i vari elementi del sistema. I *link* principali e secondari sono progettati per sviluppare uno snervamento a taglio e per questo motivo la loro lunghezza è strettamente legata alle loro caratteristiche sezionali, come ampiamente discusso nel Capitolo 3. Nel *case study* analizzato, la lunghezza di progetto dei *link* è di *4 m*, quindi le anime delle travi a doppio T devono essere abbastanza snelle da ottenere un sufficiente rapporto tra i valori di momento e taglio plastico. Ciò porta ad avere sezioni non da catalogo, nelle quali degli irrigidimenti d'anima devono essere inseriti per far fronte al problema dell'instabilità e migliorare la capacità deformativa. Il costo di questi elementi deve essere quindi tenuto in conto nell'economia degli elementi strutturali fin dalle prime fasi del progetto. Per quanto riguarda le connessioni tra i vari elementi, la complicazione sta in un layout tutt'altro che convenzionale e ripetitivo che porta ad un elevato livello di dettaglio per ogni connessione, fattore di ulteriore aumento dei costi

È da evidenziare quindi che la metodologia di ottimizzazione finora proposta non ha come obiettivo principale quello di un risparmio nella realizzazione della struttura, ma quello di ottenere un comportamento strutturale efficiente. Per sistemi strutturali semplici o tradizionali, come le strutture reticolari, l'ottimizzazione strutturale conduce indubbiamente ad un risparmio in termini di materiale utilizzato, definendo una geometria e una distribuzione di aree ottimale. Per strutture più complesse invece, come quella del caso studio, questo risultato passa in secondo piano, come evidenziato, poiché efficienza strutturale e valore architettonico diventano gli obiettivi principali. La struttura oggetto di studio risulta essere un edificio unico e di estremo valore, per il quale un costo addizionale, richiesto per creare caratteristiche prestazionali e valenza architettonica, risulta ampliamente giustificato. Quindi l'uso dell'ottimizzazione strutturale per questa tipologia di strutture non ha la pretesa di portare alla realizzazione di progetti più economici, ma è strumento indispensabile per sfruttare al meglio gli elementi strutturali, sia dal punto di vista strutturale che architettonico.

Concludendo, vengono lasciati alcuni spunti per un lavoro futuro riguardanti il software *OPTimizer* e una possibile metodologia di ottimizzazione che consideri anche il comportamento plastico delle strutture.

Riguardo il software *OPTimizer* è suggerito uno sviluppo in merito alla possibilità di considerare più materiali diversi nel processo di ottimizzazione. Nell'interfaccia *dell'OPTimizer* è infatti possibile impostare e gestire materiali differenti, con le relative proprietà e costo per unità di volume, permettendo al processo di ottimizzazione di ricavare la distribuzione ottimale di area tra gli elementi realizzati con materiali diversi, in funzione della loro efficienza, definita come il rapporto tra il modulo elastico ed il costo, settati nella funzione *cost manager*. In questo modo è possibile sfruttare al meglio gli elementi più economici, dove con il temine economico si intende sia efficienza strutturale che relativo costo. Travi in acciaio ed elementi in calcestruzzo, come il *core,* potrebbero così essere inseriti contemporaneamente nel processo di *sizing* e quindi con la funzione *cost manager* verrebbe indicata la migliore distribuzione di materiale, acciaio e calcestruzzo, in tutti gli elementi della stessa struttura. Ovviamente assume un ruolo fondamentale il "giudizio ingegneristico" nel definire limiti e campo di applicazione in cui questa funzione del programma risulterebbe utile e efficace. Infatti, se applicata indistintamente su tutti gli elementi della struttura senza alcun vincolo, la funzione *cost manager* "obbligherebbe" *l'OPTimizer* a concentrare indistintamente più materiale in elementi in calcestruzzo, a causa del suo minor costo, portando a distribuzioni di materiale sproporzionate e irrealizzabili, con ad esempio *core* di dimensioni eccessive. Si rende indispensabile in questo caso definire degli intervalli di valori di area che ogni elemento può assumere, in base a considerazioni di carattere pratico e di fattibilità geometrica. Specialmente per quanto riguarda le sezioni composte (CFT – *concrete fill tube*), questa funzione aiuterebbe a definire il rapporto ottimale tra acciaio e calcestruzzo nella stessa sezione, in relazione alla sua influenza sul comportamento strutturale globale della struttura.

Collegandosi a quest'ultimo aspetto, sarebbe utile sviluppare il software in modo che sia possibile "leggere" direttamente da *Etabs* le sezioni composte (CFT), attraverso l'API (Advanced Program Interface), funzionalità che per ora deve essere implementata dall'utente, definendo manualmente ogni sezione composta direttamente nell'*OPTimizer*.

Una considerazione può essere proposta riguardo al "dialogo" tra software commerciali ed in-house. Sebbene non siano complesse e non richiedano molto tempo, alcune operazioni devono essere svolte manualmente nel definire le impostazione nell'interfaccia *dell'OPTimizer*. Inoltre, dopo il processo di ottimizzazione, le nuove sezioni ottenute devono essere inserite manualmente in un nuovo modello su *Etabs*. Questi passaggi potrebbero essere implementati, sviluppando il software in-house, in modo da rendere l'intero processo più rapido e intuitivo per l'utente. Il processo di *sizing* potrebbe così essere integrato con *Etabs*, o con altri software di analisi strutturale, in modo da permettere all'utente di lavorare in una singola interfaccia sia sulla modellazione strutturale che nel *sizing*.

Tutte le ipotesi e le considerazioni svolte fin ora considerano una divisone netta tra analisi in campo elastico e in campo plastico: l'analisi strutturale in campo elastico è l'ipotesi con cui effettuare un'ottimizzazione topologica o di *sizing*. Nonostante ciò, la plasticità rimane un argomento molto importante nella progettazione di edifici alti e per soddisfare le sempre più numerose applicazione delle tecniche di ottimizzazione strutturale, si presenta sempre più necessaria un'integrazione tra comportamento elastico e plastico nella stessa metodologia ottimizzazione.

Nella metodologia di analisi proposta nell'elaborato, il comportamento elastico e plastico sono stati studiati separatamente, all'interno della stessa struttura, e il processo di ottimizzazione è applicato solo agli elementi che rimangono in campo elastico. Il comportamento duttile è quindi studiato solo per gli elementi esclusi dal processo di ottimizzazione. L'obiettivo di includere elementi dal comportamento non lineare nel problema di ottimizzazione dovrebbe essere approfondito per trovare nuove metodologie che consentano un design in campo elasto-plastico. L'aspetto più importante riguarda le iterazioni che saranno richieste per l'introduzione di questo tipo elementi. Nuove metodologie potrebbero quindi essere esplorate per condurre ad un’ottimizzazione strutturale in campo elasto-plastico già a partire dalla prime fasi della progettazione.

1

INTRODUCTION

Lateral resisting frame systems are one of the most important structural issues in the design of tall buildings. The aim of this research is to study a hybrid solution for a lateral resisting frame system to combine the stiffness properties of an optimized braced frame with the ductile behavior of fuse elements (links). Through structural optimization theory it is possible, in the initial phase of the design, to obtain the best frame layout for a given lateral loading condition and for a given constraint situation; the next step is to introduce a series of ductile links in order to provide the structure with localized dissipative capacity for ductile behavior. With the interaction of these two different structural systems it is possible to obtain a structure that can comply with the very strict performance requirements for high-rise buildings, both in terms of structural efficiency and esthetical beauty.

It is necessary to initially study these two structural systems separately, understanding their properties with positive and negative aspects and then have them work together to take advantage of their interaction, thus creating a structure stiff enough in service life condition and ductile under rare seismic event loading condition.

In the late 19th century, because of the arising structural challenge of new tall buildings, the effectiveness of bracing frames in resisting lateral forces was widely recognized. Most of the structures developed in early tall buildings were steel frames with diagonal bracing of various configurations such as K, X and eccentric;

in these constructions diagonals were generally embedded within the building core, usually located in the interior part.

A major departure from this design approach occurred when braced tubular structures were introduced in the late 1960s, when designers discovered the importance and the need of a dual system able to carry gravity loads with the inner concrete core and to sustain lateral loads with an exterior steel frame. In fact, tall buildings are always prone to large displacements, necessitating the introduction of special measures to contain them: one of the best examples of diagonal tapered trussed tube was the 100-story tall John Hancock Center in Chicago (Figure 1), designed by Fazlur R. Khan. Its most extraordinary feature is undoubtedly the fully exposed X-bracing on the façade. The X-bracing is continuous along each face, in order to maximize its structural effectiveness, and it is connected to the columns, which allows loads to be transferred from bracing to columns and vice versa. Beams are present at the levels where X-bracing intersects corner columns so that the bracing could redistribute gravity loads among the columns. The exposed structure of the John Hancock Center appears thin and light and at the same time it retains a look of strength and stability.

Figure 1: John Hancock Center in Chicago, IL (image courtesy of Skidmore, Owings & Merrill LLP)

In that historic period, structures as the one mentioned above came to being and were developed, from structural concepts that started to consolidate, till being clearly expressed in the building architecture.

Sears Tower in Chicago, Figure 2, is another clear example of the tube-in-tube structural systems in the design of a tall building. In this case the structure is composed of nine framed tubes to make a bundled tube, with belt and outriggers trusses at different levels, essential for the structural performances of the building.

Figure 2: Sears Tower in Chicago, IL

Conventional braced frame structures like those mentioned before have been developed till diagrids started to appear as an innovative structural system.

In diagrid structures, almost all the conventional vertical columns are eliminated [1]; this is possible because the diagonal members in diagrid structural systems can carry gravity loads as well as lateral forces due to their triangulated configuration, whereas the diagonals in conventional braced frame structures carry only lateral loads. Compared with conventional framed tubular structures without diagonals, diagrid structures are much more effective in minimizing shear deformation because they carry shear by axial action of the diagonal members, while conventional framed tubular structures carry shear by the bending of the vertical columns. For this reason diagrid structures do not need shear resistant cores because shear can be carried by the diagonals located on the perimeter.

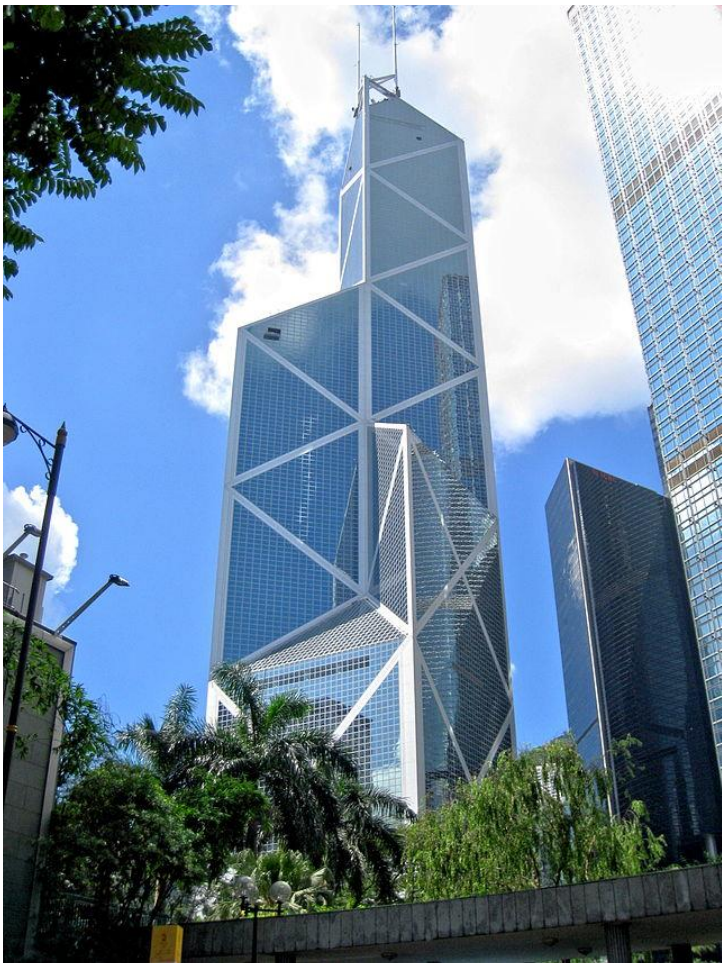


Figure 3: Bank of China in Hong Kong

The Bank of China Tower, built in 1990 in Hong Kong (Figure 3), is an example of a diagrid structures where the static role of the diagonal system appears distinctly. The lateral stiffness of diagrid structures is desirable not only for static loads but also for dynamic loads, which generate responses in both the windward and across-wind directions. In most cases, the lateral motion in the across-wind direction due to vortex shedding is much greater than the motion in the windward direction.

Beside the indisputable static advantage in using diagonal patterns, some serious aspects need to be stressed out with respect to the connection between elements like diagonals and columns or beams. Indeed, constructability is the main concern in this kind of structures because of the particular and specific details that especially the joints of diagrid need: prefabrication of nodal elements is essential. Prefabrication also reduces the costs of the structure which in this configuration tends to be more expensive with respect to the conventional orthogonal structures.

In the past few years the most valuable characteristic of diagrid structural systems turned out to be the possibility of achieving both structural efficiency and architectural significance at the same time. If on one side the structural importance of diagonals was well recognized from the beginning, on the other side their aesthetic potential was not immediately explicitly appreciated. The design of such systems is traditionally based on diagonal braces arranged according to a 45 or 60 degrees angle and variations in the range between these two angles. However, in the past few years, there have been studies to identify the optimal bracing angle and the parameters affecting such angles [1] using tools like structural optimization, thanks to which it is possible to explore optimal bracing layouts to maximize structural performance while minimizing material consumption.

Therefore, a new concept in the design of tall buildings was born: the main feature of this new resisting systems is their capability to conjugate structural efficiency and architectural beauty at the same time. This correlation between the structure and the aesthetical functions leads to design unique buildings, structurally efficient but also elegant and appealing like the one shown in Figure 4.

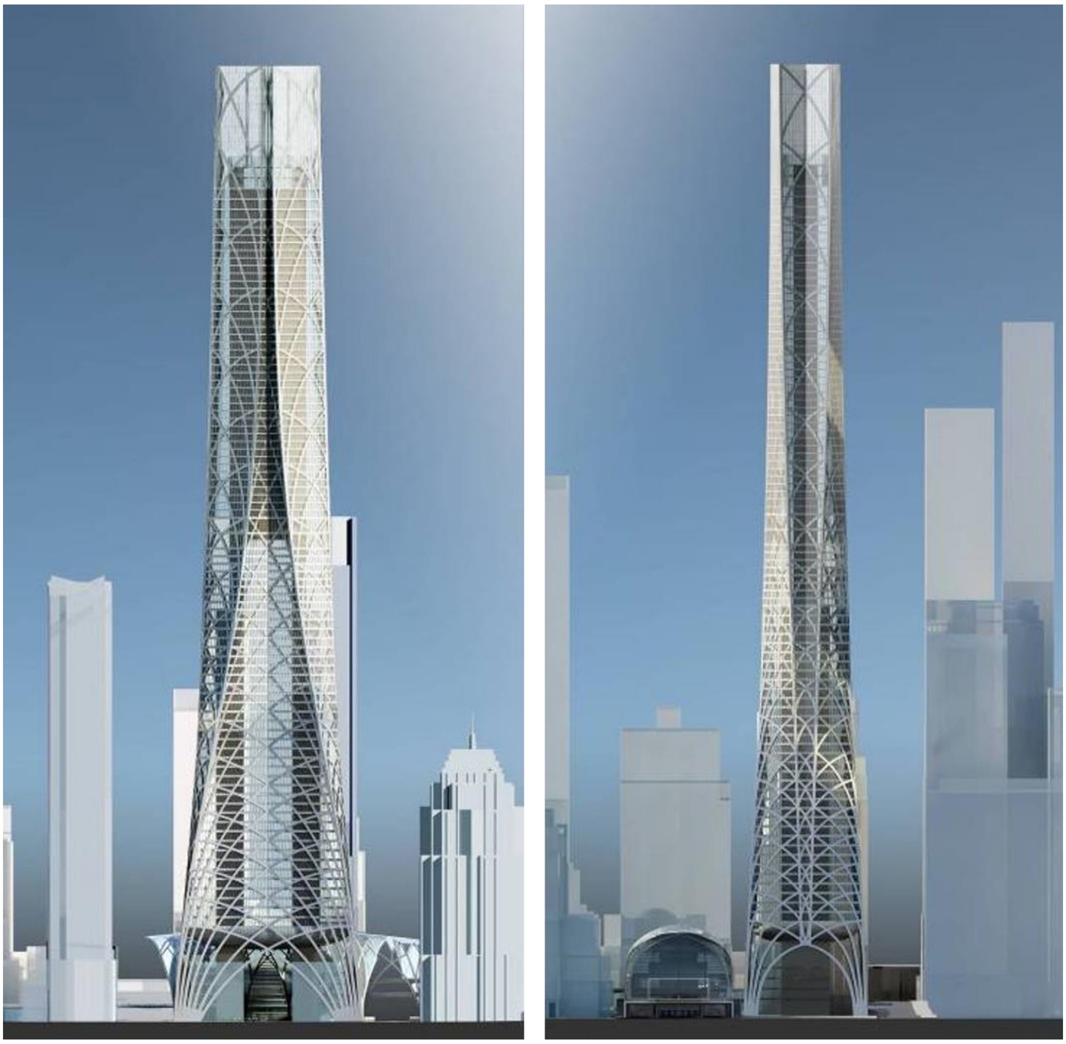


Figure 4: Example of a designed building that conjugate structural efficiency and aesthetical beauty (image courtesy of Skidmore, Owings & Merrill LLP)

Another important aspect to be considered is that in the design of lateral resisting systems for high-rise buildings, the system is considered to behave essentially elastically. This is possible where the governing load condition is the wind action, while the seismic loading condition of the particular site in which the structure is built is not so high to be the predominant condition.

The situation is completely different when the designer has to deal with high-rise buildings in very high seismicity. Especially in the last years, many new tall and super-tall buildings have been designed in areas such as China, Japan and California, where earthquake-induced forces are the predominant actions to be considered in the design phase. In designing high-performance structures subjected to important seismic loads, the structure should be able to dissipate a certain amount of energy in order to sustain a big seismic event without collapsing. If the structure had to resist these very severe seismic forces with an elastic system only, in observance with the code's limits, it would have very big section members in the lateral frame system, physically impossible to realize, both in terms of constructability and in terms of costs. Therefore, it is necessary that the entire system is modified making it able to dissipate energy.

The energy dissipation mechanism can be achieved through the insertion, in the main braced frame configuration, of particular structural devices called fuses. The "structural fuse concept" is to design sacrificial elements that will undergo large plastic deformation to dissipate seismic energy while preserving the integrity of the other structural components. This strategy has been already used in other structural applications in which the sacrificial element, after its deterioration caused by a seismic event, can be replaced without having effects on the other components of the structure and without interrupting its functionality.

This behavior, in which elastic and dissipative elements work together inside the same structure, can be found in simple structural systems like the eccentrically braced systems (EBF): these systems provide lateral stiffness and ductile behavior due to a fuse element, generally a W-shape link beam, which becomes sacrificial element. Many researches have been done in the past 40 years [2], [3] about EBF systems and their structural properties, in particular improving the ductile behavior of the link modifying its geometrical and sectional properties.

A mixed-frame system that holds both stiffness and ductile behavior, created combining EBF systems concepts with the braced frame systems ones, can be defined as Tied Lateral System. In the case study here presented (Figure 5) the combination of such a system with the topic of the structural optimization and architectural value has led to the design of a structure where aesthetical beauty and structural efficiency are joined together in the definition of a new building concept.

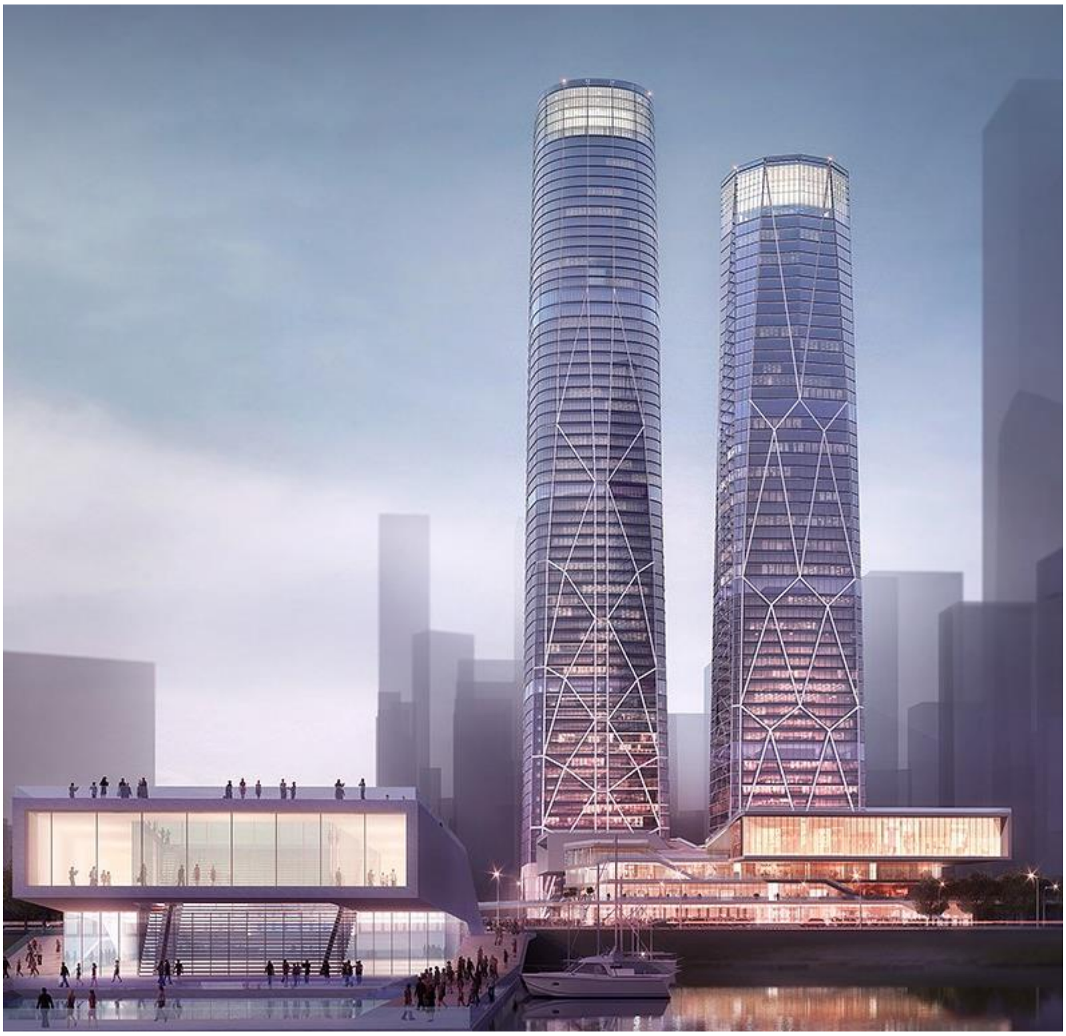


Figure 5: Case study rendering on the left (image courtesy of Skidmore, Owings & Merrill LLP)

2

STRUCTURAL OPTIMIZATION

This chapter covers the optimization techniques developed in the field of structural engineering applied to the preliminary design of tall buildings. Size and shape optimization, genetic algorithms and topology optimization are briefly illustrated with their advantages and disadvantages, investigating also the use of continuum and discrete elements in the optimization of lateral resisting systems. Alongside these concepts, the energy based design method for the design of lateral systems is widely discussed focusing on the mathematical formulation and validating it through simple structure examples. The aim is to show the large applicability of optimization methods in the structural design world.

2.1 Introduction

The objective of structural optimization is to maximize the performance of a structure or a structural component. Limited material resources, cost saving and the ever increasing need to achieve high performances lead to an optimal structural design: optimal design is defined as the best feasible design that satisfies the best performance criteria. With advances of new computer technologies, structural optimization has the potential to become an automated design tool for practicing engineers in civil engineering industries.

Since 1960, many books and papers have been published on the theoretical background and potentiality of structural optimization. Only in the last years practical optimization tools have been developed, while first researches focused mainly on the theoretical aspects of structural optimization. Among these new tools, size optimization, shape optimization, genetic algorithms, topology optimization and others were developed, as studied by M.P. Bendsoe and O. Sigmund [4]. Here follows a brief description of each of these quoted techniques, useful to understand the way of thinking in approaching the structural optimization world.

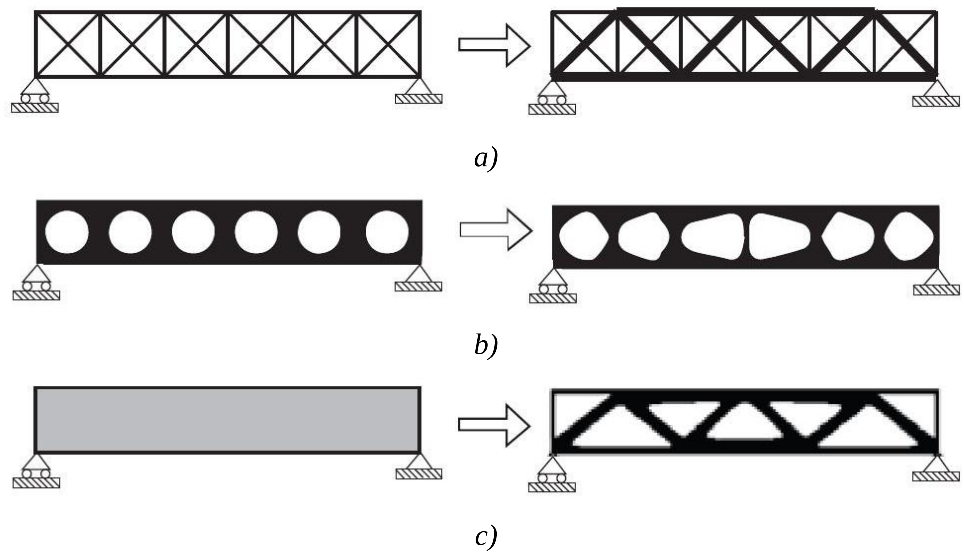


Figure 6: Sizing optimization a), shape optimization b) and topology optimization c) (image is reproduced from [4])

Size optimization (Figure 6a) is used to find the optimal area value that satisfies a certain target parameter; constraints, shape and position of each element cannot be changed in this process. In this work, size optimization has been used applying the energy based design method for lateral systems design introduced by Baker in 1992 [5]. In the case study presented in Chapter 5, the elements configuration was already defined, as will be showed, and this kind of optimization has been rather handily in finding the right proportions in terms of elements areas using as target parameter the tip displacement of the structure.

A different technique is the shape optimization, which is focused on the distribution of material through the design domain. Without changing topological properties, like the numbers of holes in the example of Figure 6b, it re-shapes the material layout leading to an optimal solution. The design variables can be the thickness distribution along structural members, the diameter of the holes, the radii of fillets or any other parameter.

Yet another technique is the surface optimization [6], in which the shape of a building can be optimized using topographical surface optimization based on different objective functions, such as minimum compliance, minimum tip deflection, optimal frequencies etc. Structural aesthetics, geometry, masses, optimal view angles could be the constraints of the problem. This optimization is conducted with a combination of commercially available codes and custom written programs: the software starts from a generic initial, arbitrary shape that is iteratively modified through the optimization process until the best shape for the considered objective is obtained. In the example shown in Figure 7 the constraint is the radii of the floor plates at various elevations that control geometrically the process. Other parameters could be instead be limited to code or user set values such as the height of the building, site constraints, the base diameter and the overall internal volume.

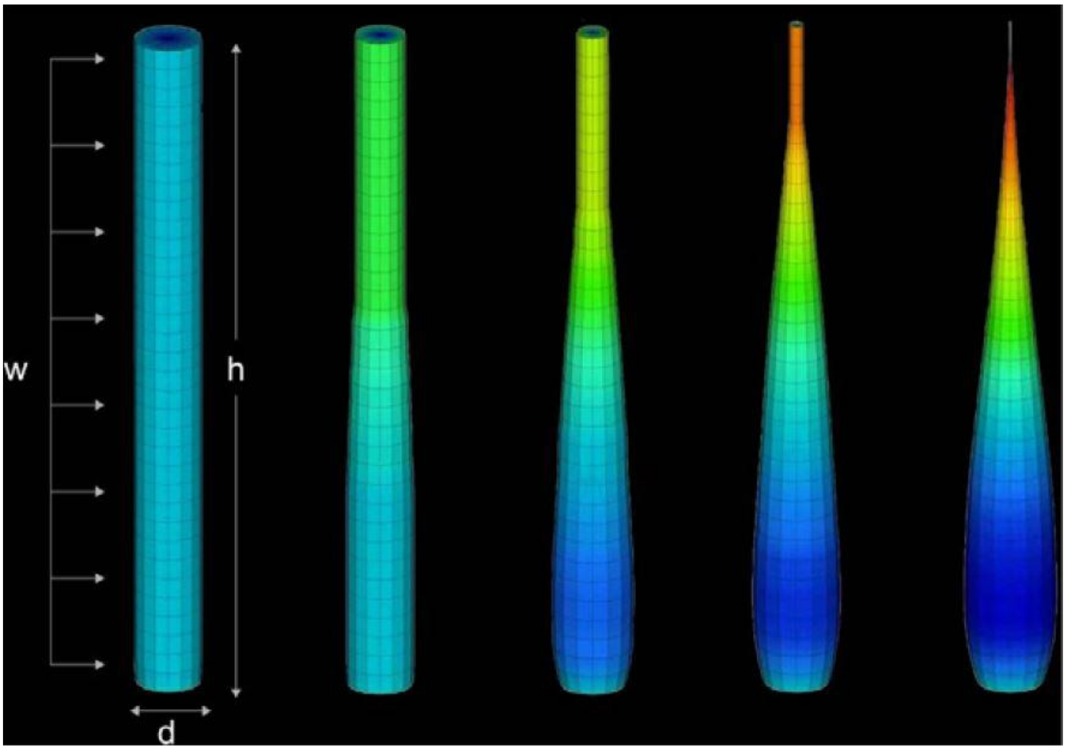


Figure 7: Shape optimization (image is reproduced from [6])

In the example, proposed by Beghini A. at al. [6] in Figure 7, a uniform lateral load is applied on the initial shape and the building surface is iteratively modified to minimize the top displacement that is set as the structural goal.

An optimization tool commonly used in the design industry is the use of genetic algorithms. Principles of natural selection can be used: the design with higher fitness values has a higher probability of being selected to produce the next generation of solutions. In structural engineering terms, the best solution is the one which satisfies a target structural goal (maximum stiffness for instance) for a certain volume of material (as a constraint). The process starts with an initial randomly-generated population of design solutions, and this population evolves over many generations through selection of parent solutions. Then is performed a reproduction processes with the aim of improving individual designs accordingly to some measure of fitness. This fitness measure corresponds directly to the objective function of the optimization problem.

The process ends when additional iterations bring minor changes to the structural performance. The genetic algorithm can then communicate with the finite element software via custom-written codes.

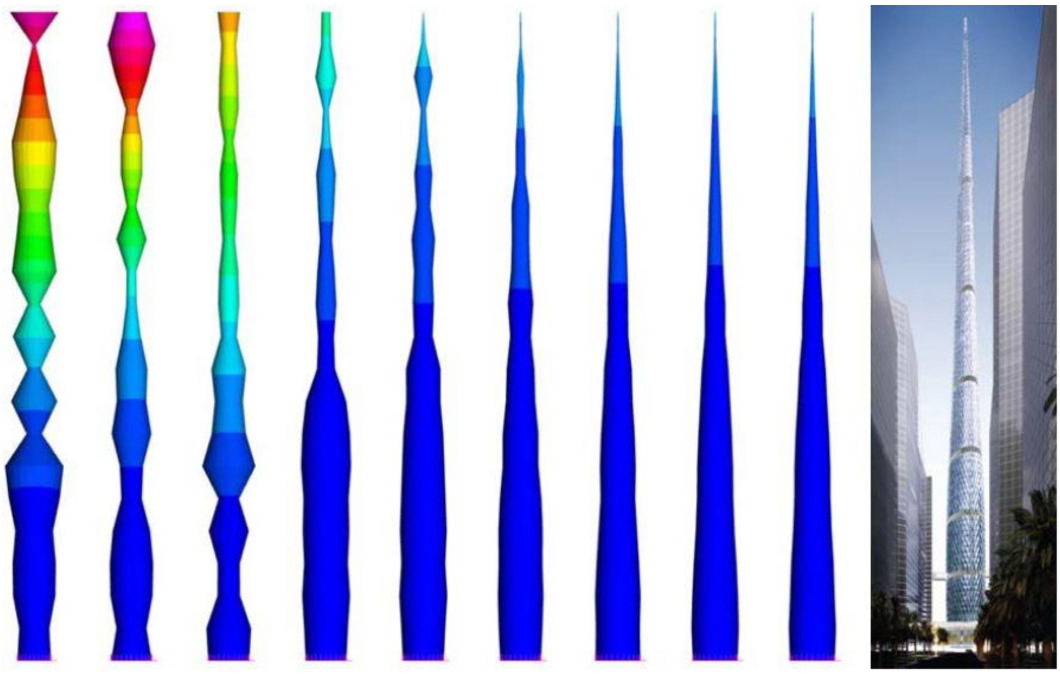


Figure 8: Genetic Algorithms optimization (image is reproduced from [6])

This process is summarized in Figure 8, from the work of Beghini A. et al. [6], where at each iteration of the analysis, the finite element model is modified according to the parameters determined by the genetic algorithm, and a structural analysis is run. The calculated structural performance based on the structural objective is then returned to the genetic algorithm for fitness evaluation.

An overall overview of the main optimization techniques has been illustrated while topology optimization will be presented in the next section. The case study will provide a deep exploration of both size and topology optimization and, regarding the size optimization, an in-house software will be used to size the lateral system of a complex structure.

2.2 Topology optimization

In this section we discuss topology optimization by summarizing the main results presented in details in Beghini L.L. [7], in Stromberg L.L., Beghini A. et al. [8], [9].

The purpose of topology optimization is to find the optimal layout of a structure, within a specified region, for a specific object function (target deflection, compliance, etc). With respect to the other techniques explained above it turns out to be a more wide and general method that can deal with different aims. The only known quantities in the problem are the applied loads, the constraints conditions and the whole volume of the structure to be realized. In addition to these known parameters, some other additional information could be given such as the location and dimensions of apertures or solid areas to be maintained during the design. In this methodology the physical size, shape and connectivity of the structure are unknown, so the feasible solutions can have any shape, size or connectivity. The finite element method (FEM) is here applied by dividing a design domain into several small pieces, finite elements, and each element is used to represent the ideal design, as a pixel of an image, by containing a density that is either solid (black) or void (white).

The result of topology optimization is then a pattern of small pieces inside the domain, combined to create the final structure configuration. This new layout is relevant, first of all for the engineers, because it allows to recognize the main path of forces flowing inside the structure. If topology optimization is performed in a tall buildings design, the geometry configuration obtained would become the layout of the main resisting system. In the case study, the lateral braced frame layout is obtained through topology optimization and, as it will be explained in Chapter 5, good structural performances are achieved.

Besides structural advantages, engineers have the possibility to integrate architectural components inside this process and consequently develop an interactive dialogue with architects. The possibility to propose a structure that can combine structural efficiency and architectural value is undoubtedly the key point of topology optimization. Very often, the history has shown how buildings that have left a mark in the fields of architecture and engineering are those in which aesthetic and efficiency are joined together.


Figure 9: Broadgate Exchange House in London, UK; Bank of China Tower in Hong Kong

In figure 9 it is possible to understand better the concept expressed above looking at the exposed structure of the two buildings. Without exploring in detail each of the two structural systems it can be noticed that the engineering aspect is clearly expressed into the design of the buildings, giving them an architectural feature while satisfying pure structural principles.

This desirable collaboration between architects and engineers, working together to express both structural and architectural concepts, can then be achieved with the topology optimization and also the case study building is a terrific result of that. More generally, in the design of a building the criteria of the structural engineer may focus on tip deflection limits, lateral load resisting systems (braced frames or concrete core), sizing and placement of structural members (i.e. beam and columns) and on the simplification of the design by using symmetry and patterns, among others. On the other hand, the architect may consider a different range of criteria regarding the aesthetics of the building, such as the value of views, cladding (e.g. glass facade), incorporation of landscaping (green areas), symmetrical appearance and patterns. Though topology optimization results are guided by engineering judgment, several options can be changed in the structural context to explore different outcomes. For example, a different design space or various combinations of loads and boundary conditions can be explored.

If on one side the topology optimization gives the possibility to achieve a new elements configuration with respect to a defined target function, on the other side it is important to select the right objective function to suit the problem. In addition to that, also the field of application could be different and in the past several approaches have been proposed; for example topology optimization for braced frames, topology optimization problems for stability design (critical buckling load), natural frequency and dynamic loads.

In the conceptual design phase of a high-rise building, the majority of the objective functions are usually related to the overall stiffness/drift requirements under lateral loads: therefore, many of the decisions made during this process are related to defining the lateral system, or providing stiffness/drift control. Through the selection of the objective function and other parameters that might fit the problem being studied, the engineers can then coming to the architects with a spectrum of solutions based on these parametric studies.

Mathematical background

In topology optimization, as presented by Bendsoe M.P. and Sigmund O. [4], the optimal layout of material for a given design domain is searched in terms of an objective function. In this way the optimization problem can be formulated in his general aspect and refers to a wide number of problems. From a mathematical point of view, optimization consists on the research of stationary point in the function describing the problem. The generalized statement for the topology optimization problem can be written as follows:

$$min_d \quad f(\boldsymbol{d}, \boldsymbol{u}) \tag{1}$$

$$s.t. \quad g_i(\boldsymbol{d}, \boldsymbol{u}) = 0 \; for \; i = 1, \dots, k \tag{2}$$

$$s.t. \quad g_i(\boldsymbol{d}, \boldsymbol{u}) \leq 0 \; for \; i = k+1, \dots, m \tag{3}$$

Where $f(d,u)$ represents the function, $\boldsymbol{d}$ is the design field, $\boldsymbol{u}$ is the response (natural frequency, stress levels, ductility, ecc.), and they are related through the constraint function g_i. An example is the minimum compliance problem:

$$min_d \quad f(\boldsymbol{d}, \boldsymbol{u}) = \boldsymbol{p}^T \boldsymbol{u} \tag{4}$$

$$s.t. \quad g_1(\boldsymbol{d}, \boldsymbol{u}) = \boldsymbol{K}(\boldsymbol{d})\boldsymbol{u} - \boldsymbol{p} \tag{5}$$

$$s.t. \quad g_2(\boldsymbol{d}) = V(\boldsymbol{d}) - V \tag{6}$$

Where g_1 represents the equilibrium constraint and g_2 is the constraint on the allowable volume of material V. A continuous variation of density in the range (*0*, *1*) is applied rather than restricting each density to an integer value of *0* or *1* guaranteeing the existence of a solution in the setting. To avoid singularity in the global stiffness matrix $\boldsymbol{K}(\boldsymbol{d})$, a small parameter $\boldsymbol{d}_{min}$, is specified.

The Solid Isotropic Material with Penalization (SIMP) model, studied in [4], is commonly used to solve topology optimization problems. In this formulation, the stiffness and element density are related through a power law relation of the form:

$$\boldsymbol{E}(\boldsymbol{x}) = \boldsymbol{E}_{min} + \boldsymbol{d}(\boldsymbol{x})^p(\boldsymbol{E}_0 - \boldsymbol{E}_{min}) \tag{7}$$

Where E_0 is the Young's modulus of the solid phase of material and p is a penalization parameter to eliminate intermediate densities with $p \geq 1$. The SIMP model ensures that material properties continuously depend on the material density at each point. The penalization parameter p forces the material density towards *0* or *1* (void or solid respectively) by penalizing regions of intermediate densities (gray zones) where $\boldsymbol{d}$ assumes values in the range of *0* to *1*.

Resuming, the field of structural topology optimization can, for the sake of simplicity, be separated in two classes of methods. In the first category, there are the continuum approaches where the objective is to find optimal continuum distribution. This solution provides valuable insight about the geometry of the layout of the members of the optimal structure. Because optimal continuum material distributions may or may not lead to structural systems with truss-like or beam-like elements, continuum topology is in this sense more general and not of direct application; the resulting discrete structure may be difficult to interpret. In the second category, there are the ground structure approaches wherein the objective is to find the optimal placement of discrete structural elements. In these methods, the design problem typically consists of assuming a base or ground structure with a given layout of members. The application of discrete topology optimization methods to truss-like structures has been studied by a considerable number of engineers including Mazurek, Beghini and Baker [10] [11], Michell [12], Prager [13]. Indeed a new challenge associated with the use of topology optimization for structural engineering concerns the integration with the architectural components.

2.2.1 Optimization for lateral resisting systems: combining continuum and discrete elements

Effects of wind and seismic loads on buildings increase rapidly with increases the height: stiffness and stability performance requirements are often dominant criteria in the design of tall buildings. Therefore framed structural systems are required as a main component in the design.

There are two methods to satisfy the stiffness and stability performance criteria. The first is to increase the sizes of the members, but this would produce unpractical or uneconomical designs. The second is to select a more rigid and stable lateral load resisting system to reduce the deflection and increase the stability. The design of tall buildings under lateral loads is first of all governed by system performance criteria and overall stiffness, rather than component performance criteria and strength.

Stiffness-based sizing techniques have been developed in the last years with the aim of minimizing the weight of lateral systems. Baker W. F. [5], as mentioned before, presented a sizing technique based on energy methods for lateral resisting systems in multistory steel buildings. However, this sizing technique can be applied only to lateral resisting systems with already defined and fixed topologies, which limit and influence the total effects of the sizing process in terms of final performance of the particular bracing system.

The optimal layout design for bracing systems can be achieved using topology optimization: in that way engineers can identify the optimal geometry of the lateral bracing systems in addition to minimize material usage. With this methodology, engineers are able to design the lateral system from the conceptual optimal bracing angles to the final sizing of the members.

In the studies by Stromberg L.L. at al. [8] an early attempt was made to identify optimal bracing angles using continuum approach.

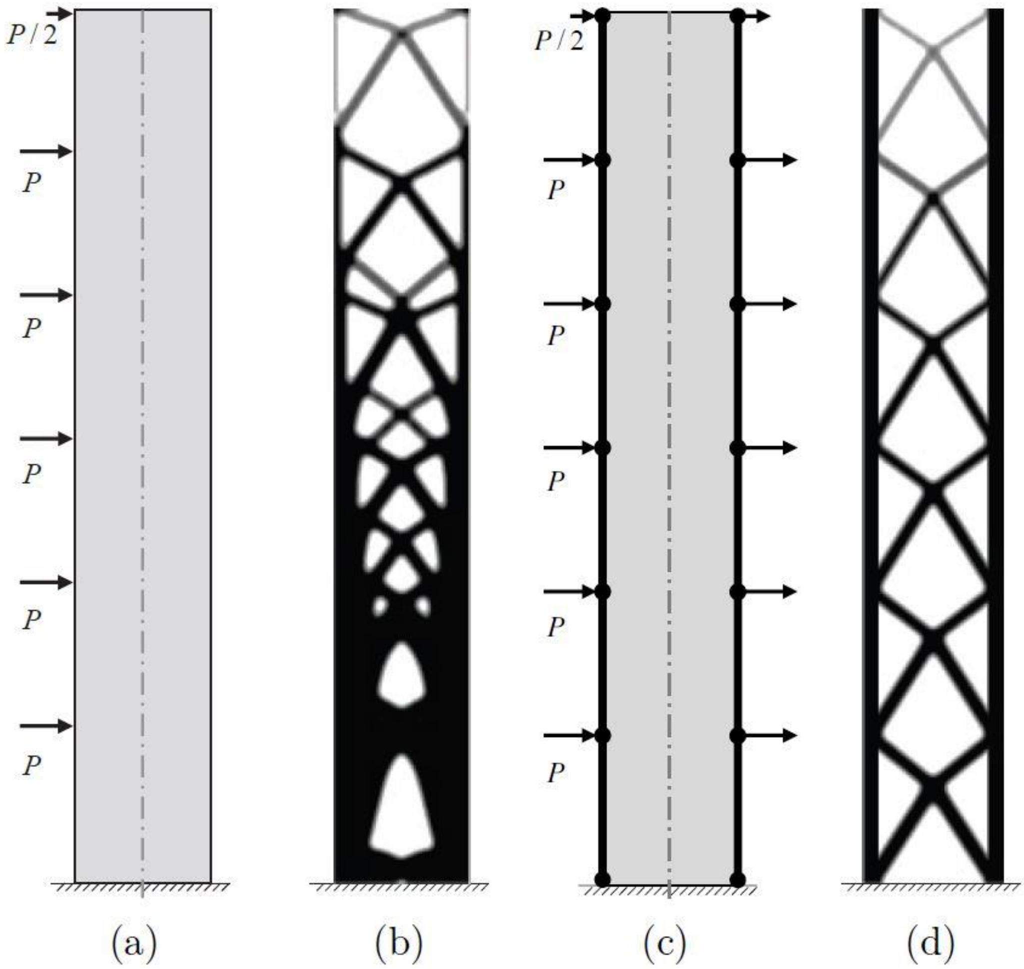


Figure 10: Comparison of optimization techniques: a) problem statement for continuum approach, b) topology optimization result using quadrilateral elements, c) problem statement for combined approach, d) topology optimization results with quadrilateral and beam elements f [8])

However, some limitations are evident in the optimization results in Figure 10b, which shows a simplification of a high-rise building subjected to lateral loads. In the example, the result shows high concentrations of material towards the extreme edge of the domain, as expected from the web-flange behavior, typical of these structures, discussed in Stromberg L.L. at al. [9]. These high-density regions of material make it difficult to visualize and determine the right bracing work points (i.e. the locations where the diagonals intersect the columns). In addition, such high concentrations may lead to incorrect flexural stiffness in the analysis of the structure. In practice, this is not realistic because in a real building columns are relatively narrow compared to the width of the building. Moreover, if the majority of material is “optimal” at the extreme edges of the domain, the structure has an incomplete diagonalization, because small amounts of material are leftover to form the diagonal members: and this fact is not possible on a realistic building design. It is necessary to introduce an additional constraint to distribute material between columns and diagonals to prevent high concentrations at the edges. The addition of discrete element as beam and column elements into the design domain, in the analysis procedure, eliminates the problem: as shown in Figure 10d, the issues mentioned are no longer of concern for structural design. The discrete column elements now give practical bending stiffness to the structure, the diagonalization is complete along the height of the structure and the bracing members are undoubtedly identified.

After the layout of the braced system is defined, the optimization techniques help lead to the final sizing of each member, process that can be used at each stage of the project process.

Baker W.F., in the Energy Base Design Method [5], derived a method to calculate the optimal cross-sectional area for a statically determinate frame to limit the tip displacement of a building under wind load to a target deflection by combining the PVW and the Lagrangian multiplier method.

The optimization techniques described previously help streamline the design decisions at various stages of a project from the conceptual definition of a braced frame layout to the final sizing of the members. Once the overall shape of the building is known, the optimal bracing layout could be established for example assuming that frame columns are arranged around the outer perimeter at a regular spacing, to ensure that the tributary areas for the columns are similar. Beams and columns would be modeled using beam elements while the space bounded by two columns and two beams would be meshed using quadrilateral elements. After the finite elements mesh is completed, the following steps can be applied in sequence in the design flow process:

- Size vertical elements (columns) according to gravity load combinations (accounting for dead, superimposed dead and live loads)

- Run topology optimization on the continuum for lateral loads combinations (accounting for wind and seismic loads)
- Identify the optimal bracing layout based on results to create the frame model
- Optimize the member sizes using the virtual work methodology
- Edit geometry and iterate if necessary

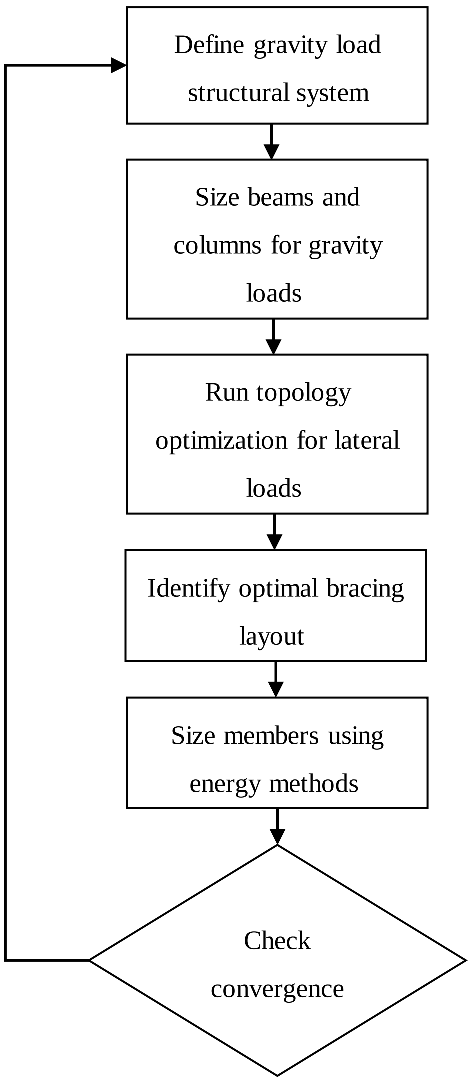


Figure 11: Topology optimization workflow (image is reproduced from [7])

The above steps indicate a potential path from a conceptual design to the final sizing of a braced frame. However, each optimization step could be performed independently depending upon the specific need of the engineer or of the architect. The process might be iterative after the geometry is edited until convergence is achieved or the requirements are satisfied.

2.3 Energy Based Design Method

The energy-based method for the design of lateral systems, proposed by Baker W.F. [5] and presented in this section, shows a way to calculate the optimal cross-sectional area for a statically determinate frame to limit the tip displacement of a building under lateral load to a target deflection. This goal is reached combining the principle of virtual work (PVW) and the Lagrange multiplier method, obtaining a structure in which the members are appropriately sized and the structural materials are efficiently distributed among the various components.

As mentioned in the introduction this sizing methodology implies that the lateral resistance system design is controlled by stiffness and not by strength because of the direct relation between stiffness and displacement: the more the target displacement is set to a small value the more the structure has to be stiff.

2.3.1 Axial members

The starting point is to size the structure so that the internal work is attained with a minimum structure volume. It is known from virtual work methods that the deflection at the top of a braced structure is given by:

$$\Delta = \sum_{i=1}^{n_{el}} \frac{n_i F_i L_i}{E A_i} \tag{8}$$

Where:

F_i = Force in a member due to lateral load case

n_i = Force in a member due to unit load case

L_i = Element length

A_i = Element area

The equation can be seen as a term, *FL/EA*, that contains the actual elongation of the individual member and a weighting function, *n*, that gives the relative influence of the individual members on the total deflection.

Since the goal is to use the least volume of material, it is useful to divide the deflection contribution of each member by the volume of each member. In this way is obtained a term which is essentially a virtual work or energy density (e) and can be viewed as a measure of efficiency.

$$e = \frac{n_i F_i}{E A_i^2} \tag{9}$$

By moving the material from one member to another, it is possible for all members of the structure to have equal energy densities. Now the aim is to minimize the deflection obtaining a structure with the minimum volume, this can be investigated using Lagrange multipliers.

The function *f* we want to minimize is the deflection:

$$f = \sum \frac{nFL}{EA} \tag{10}$$

The minimization of the deflection is constrained by the fact that the structure has a constant volume of material:

$$g = \sum AL - V = 0 \tag{11}$$

Therefore the deflection can be seen as:

$$\Delta = f + \lambda g \tag{12}$$

Substituting *f* and *g* from equations above:

$$\Delta = \sum \frac{nFL}{EA} + \lambda \left(\sum AL - V \right) \tag{13}$$

Where λ is a constant called Lagrange Multiplier.

Given that the geometry of the structure is set, the independent variables are the areas of the individual elements (A_i). Then, in order to find the minimum for the function Δ, the constrained deflection is differentiated with respect to the areas:

$$\frac{\partial}{\partial A_i} \left(\sum \frac{nFL}{EA} \right) + \lambda \frac{\partial}{\partial A_i} \left(\sum AL - V \right) = 0 \tag{14}$$

Noting that the terms f and g are continuously differentiable and g has a nonzero gradient, it can be shown that λ exists and is unique at a local extremum.

If a structure is statically determinate, n_i and F_i are constant for a given structure and for a given loading.

Equation (14) reduces to:

$$-\frac{n_i F_i L_i}{E A_i^2} + \lambda(L_i) = 0 \tag{15}$$

And then:

$$\lambda = \frac{n_i F_i}{E A_i^2} \tag{16}$$

When λ is equal for all members in a structure, the deflection is a minimum for a given volume of the structure or, conversely, the volume of structure is a minimum for a given deflection. By comparing equations, it can be seen that for a system of axial members the energy density e is in fact the Lagrange multiplier for a statically determinate structure.

Starting with a model with arbitrary cross-sectional areas, combining the deflection and the Lagrange multiplier, the optimum areas for a statically determinate truss can be found through the following steps.

The area A_i can be made explicit from equation (16) :

$$A_i = \sqrt{\frac{n_i F_i}{E\lambda}} \tag{17}$$

Substituting in equation (8) :

$$\Delta = \frac{1}{E}\sum_{i=1}^{n_{el}}\left(\frac{n_i F_i L_i}{\sqrt{\frac{n_i F_i}{E\lambda}}}\right) \tag{19}$$

Simplifying equation (19) and substituting λ from equation (16), the area of each element is given by:

$$\Delta = \frac{\sqrt{\lambda}}{\sqrt{E}} \sum_{i=1}^{n_{el}} \left(L_i \sqrt{n_i F_i} \right) \tag{22}$$

$$(A_i)_{req} = \frac{1}{\Delta_{req} E} \sqrt{n_i F_i} \sum_{j=1}^{n_{el}} \left(L_i \sqrt{n_i F_i} \right) \tag{23}$$

Where:

Δ_{req} = Target deflection

n_i = Virtual force in member *i*

F_i = Real force in member *i*

The result shows how, once the structure geometry is set and the loads are defined, there is a linear relation between the element area and the target deflection. Without further calculation, if the target deflection change also the area values change, but the proportion between the element areas remains the same. For example if the target displacement is set to the half of the one required before, the area values will simply double.

Example

In order to show a simple application of the method above a structure is chosen with the following loads and geometry:

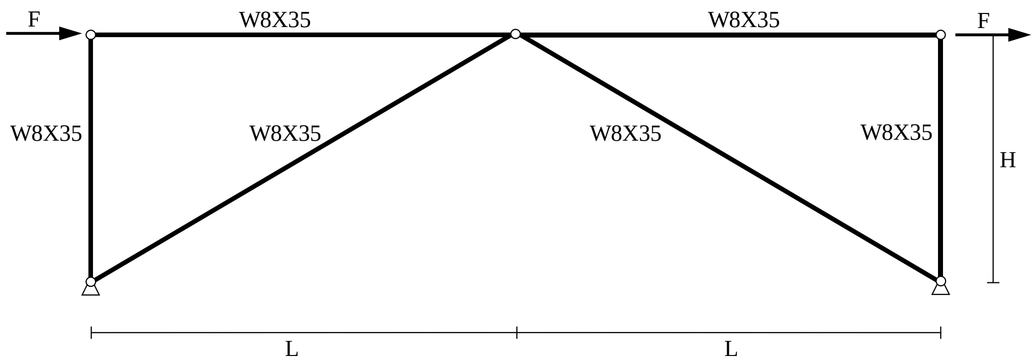


Figure 12: Geometry and section of the example structure

Data:

$L = 204\ in$

$H = 120\ in$

$F = 100\ kip$

The structure is statically determinate, two lateral point loads *F* of the same intensity are applied at the top of the brace with height *H* and total length *2L* and the sections are W-flange with an arbitrary area equal for all the elements.

In Figure 13 is represented the stress distribution under the real load with the member that work only in tension (blue fill) or compression (red fill)

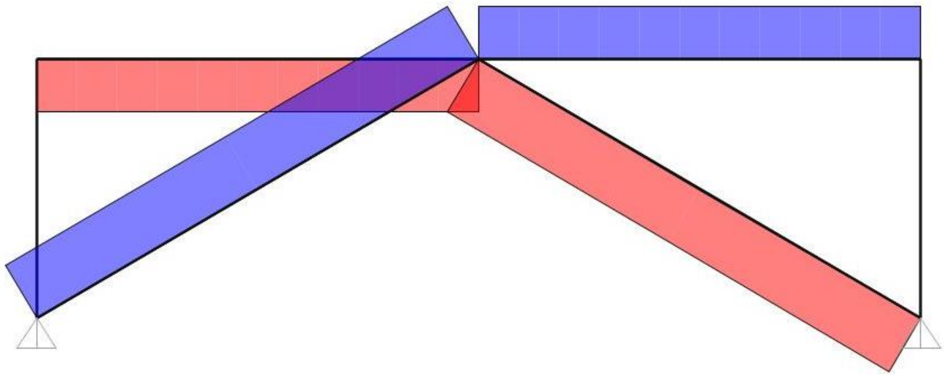


Figure 13: Axial stress under real load (tension in blue, compression in red)

On the other hand in Figure 14 the stress distribution is shown with the unit load applied at the top right end being the displacement evaluated with respect to that point.

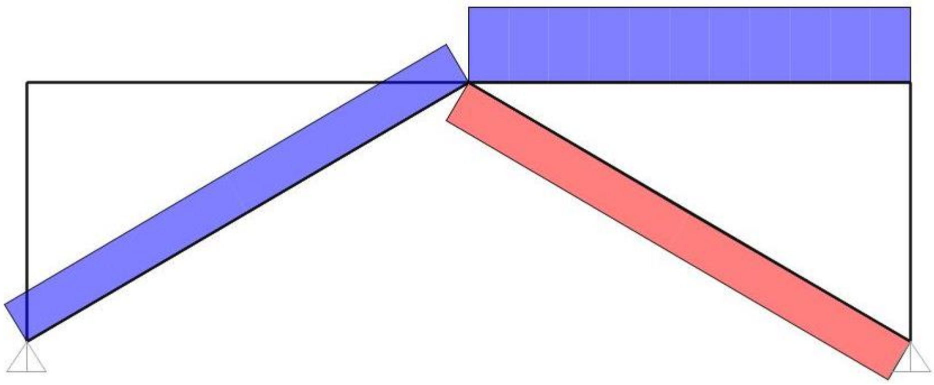


Figure 14: Axial stress under unit load (tension in blue, compression in red)

The displacement of the structure at the top under the given loads is *0.35 in*. The target deflection is set to *0.05 in* and the formula for the areas optimization is applied obtaining the following result for areas and energy densities:

Table 1: Optimized areas result

***A**$_{i, opt}$* [in^2]		***e**$_i$* [in^{-2}]	
A1	0	e1	0
A2	0	e2	0
A3	82	e3	1E-06
A4	0	e4	0
A5	67	e5	1E-06
A6	67	e6	1E-06

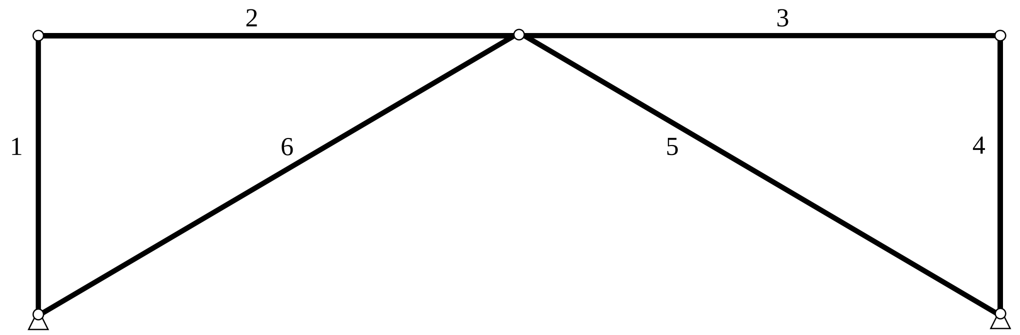


Figure 15: Members numbering of the example structure

Now the new sections are chosen from the list of the available ones and the target displacement is calculated with these new sections; the sections in which the area is zero maintain the same initial sections except for element *2* that is made equal to element *3* to have a symmetric structure.

Table 2: Final optimized areas

SECTIONS		***A**$_{i, final}$* [in^2]	
elem 1	W8X35	A1	10.3
elem 2	W36X282	A2	82.9
elem 3	W36X282	A3	82.9
elem 4	W8X35	A4	10.3
elem 5	W36X231	A5	68.1
elem 6	W36X231	A6	68.1

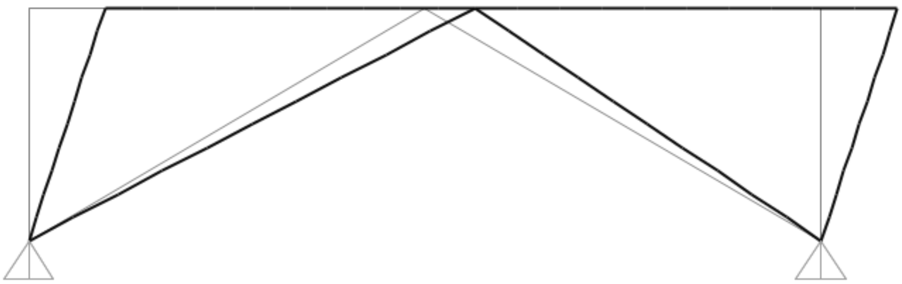


Figure 16: Example structure deformed shape

The displacement at the top right angle is *0.492 in* that satisfied the target deflection required of *0.05 in*.

2.3.2 Flexural members

In many cases the axial contribution is not enough to correctly describe the structure and to obtain the optimal areas for all the members. With an approach similar to the one explained in the previous section, the flexural members can be widely studied.

The deflection contribution of flexural deformation is given by

$$\Delta = \sum_{i=1}^{n_{el}} \frac{L_i \overline{M}_i}{EI_i} \tag{24}$$

Where:

$\overline{M}_i = \int_0^l Mm\, dx$

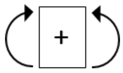
M = Moment in a member due to lateral load case

m = Moment in a member due to unit load case

The integral is well known for elements with the moment applied at the starting (s) and final (f) end nodes, as is usually the case of lateral analysis of multistory buildings, and is given by:

$$\overline{M} = \int_0^l Mm\, du = \frac{1}{6}\left[M_s\left(2m_s + m_f\right) + M_f\left(2m_f + m_s\right)\right] \tag{25}$$

The sign of the bending moment for each element is taken positive with respect to the following scheme:



As was done with axial members, a virtual work density can be calculated by:

$$e = \frac{\bar{M}_i}{EI_i A_i} \tag{26}$$

In the case of the flexural action in a generic member, as will be shown below, making the energy densities equal for all members, for a statically determinate structure, will lead to an optimum material distribution only for certain types of cross section while for all others cross sections is approximately minimal.

This can be shown again by using the Lagrange multiplier approach. As was done for axial members the constrained displacement can be written as:

$$\Delta = \sum \frac{L\bar{M}}{EI} + \lambda \left(\sum AL - V \right) \tag{27}$$

The local extremum is found by differentiation:

$$\frac{\partial \Delta}{\partial A_i} = \frac{-L_i \bar{M}_i}{EI_i^2} \frac{dI_i}{dA_i} + \lambda L_i \tag{28}$$

Or:

$$\lambda = \frac{\bar{M}_i}{EI_i^2} \frac{dI_i}{dA_i} \tag{29}$$

As equation (29) shows, the problem is to evaluate the derivative of the inertia with respect to the element area: for elements with rectangular section of constant depth (h) and variable width (w) the derivative can be expressed through the radius of gyration (r) while for other sections a linearization is needed:

Whit rectangular shape:

$$I = \frac{1}{12} wh^3 = \frac{1}{12} Ah^2 = r^2 A \tag{30}$$

$$\frac{dI}{dA} = r^2 \tag{31}$$

Therefore the expression of λ matches the energy density (26):

$$\lambda = \frac{\bar{M}_i}{EI_iA_i} \tag{32}$$

The linearization of the inertia is necessary considering other types of section shapes like T-shape or W-flange shape; in general for rolled US steel shapes, the moment of inertia can be expressed as:

$$I = a + bA \tag{33}$$

That leads to:

$$\frac{dI}{dA} = b \tag{34}$$

And

$$\lambda = \frac{\bar{M}_i b_i}{EI_i^2} \tag{35}$$

Which is somewhat different from the energy density calculated before.

In order to obtain the equation for the optimal elements areas, it is useful to define λ in terms of areas, radii of gyration and correction factors:

$$\lambda = \frac{\bar{M}_i \alpha_i}{Er_i^2A_i^2} \tag{36}$$

Where α is a correction factor defined as:

$$\alpha_i = \frac{1}{r_i^2}\frac{dI_i}{dA_i} \tag{37}$$

For rectangular shape, $\alpha = 1$, while for rolled shape α is calculated and linearized. A reasonable value for the design of commonly used shapes in strong axis bending is $\alpha = 1.3$.

Linearization for W-flange is shown in the next graph (Figure 17). As can be seen a linearization is needed for each kind of section type and for a few of them a division in the linearization inside a single series has to be made.

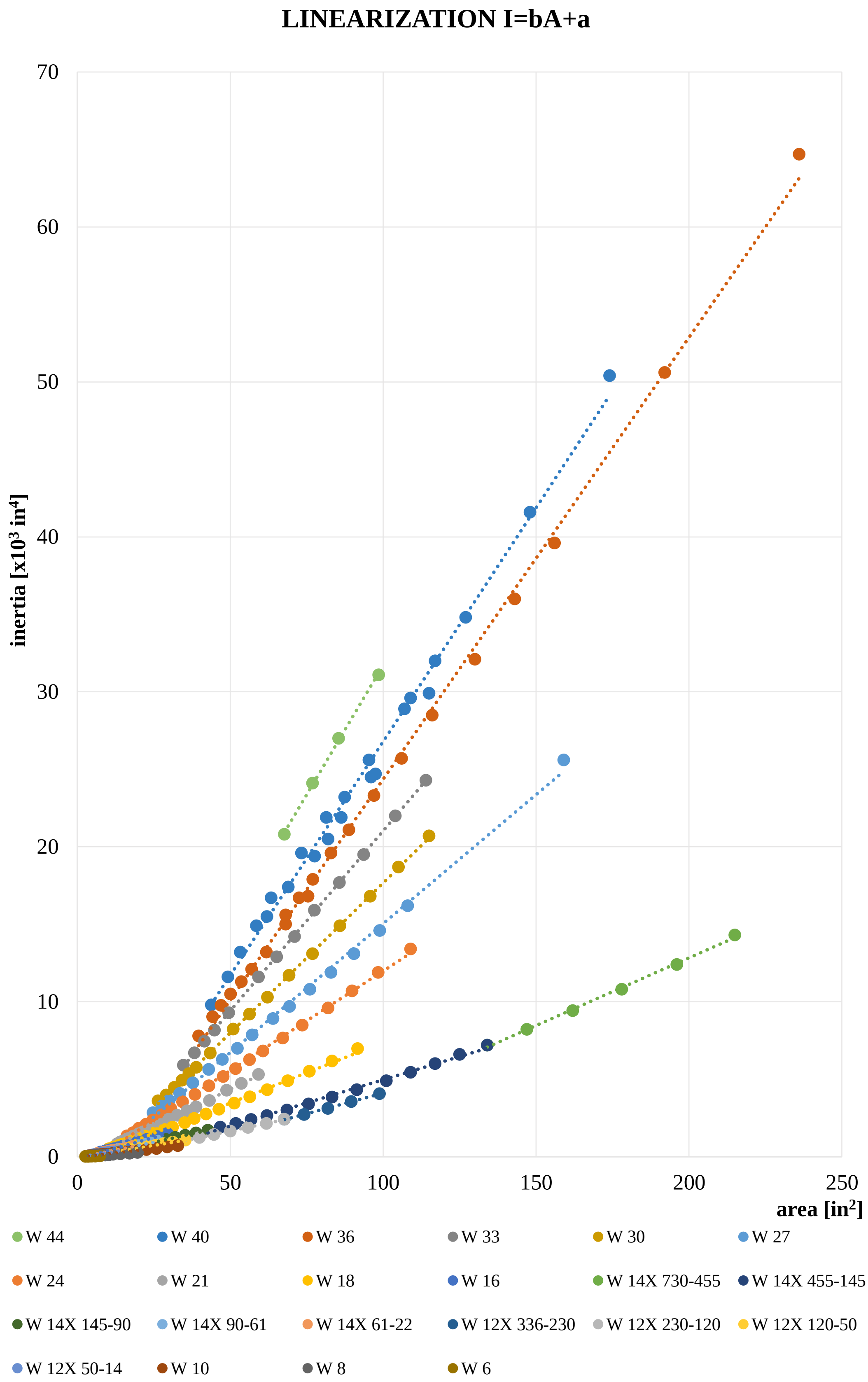


Figure 17: Inertia-area linearized relation

As was done for axial members, the optimal cross-sectional area for a statically determinate flexural member can be determined from:

$$(A_i)_{req} = \frac{1}{\Delta_{req}E}\frac{\sqrt{\overline{M}_i}\sqrt{\alpha_i}}{r_i}\sum_{j=1}^{n_{el}}\left(\frac{L_j\sqrt{\overline{M}_j}}{\sqrt{\alpha_j}r_j}\right) \tag{38}$$

In the formula above the radius of gyration is assumed to be constant for a member as the members change size, this is a reasonable assumption as long as each member stays in the same series.

<u>*Section choice*</u>

The main point to be discussed, related to the choice of the sections, is the fact that the optimization process gives, as a result, a value of area and, at the same time, different values of inertia can be associated to that value of area. A possible solution could be to apply the linear relations used between inertia and area, but this implies that the series of profiles initially arbitrarily chosen would have to remain the same. In addition to this all the area values found should be in the same series to be described by a single line

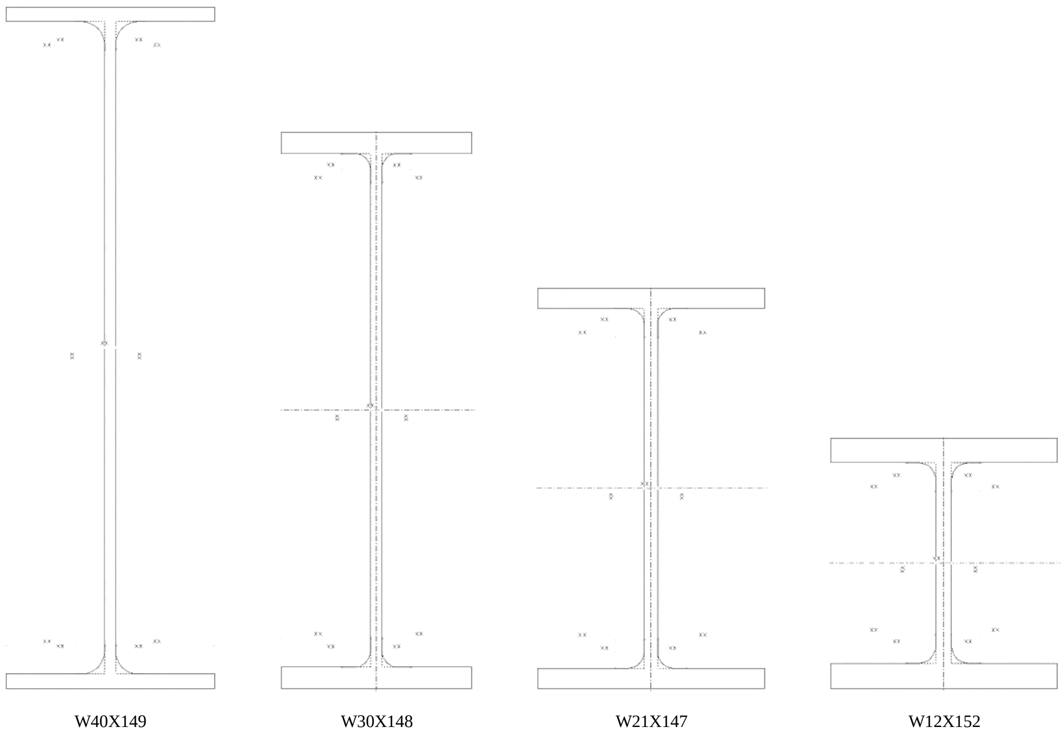


Figure 18: Different shape associated to sections with same area

Table 3: Chosen sections

Name	**Area** (in^2)	**Inertia** (in^4)
W40 × 149	43.8	9800
W36 × 150	44.2	9040
W33 × 152	44.8	8160
W30 × 148	43.5	6680
W27 × 146	42.9	5630
W24 × 146	43	4580
W21 × 147	43.2	3630
W18 × 158	46.3	3060
W14 × 145	42.7	1710
W12 × 152	44.7	1430

Table 3 shows the different values of inertia that can be associated to a small range of area values and in Figure 18 the comparison between the different shapes of different series shows how important the choice of the section associated to a cross-sectional area is. Being the flexural component dependent on the inertia a correct way to solve the problem is to iterate after the first choice of sections. In this way the distribution of stress change according to the new inertia values and in the further choices the section has to be selected as closest as possible to the previous one in order to be described by the same linearization coefficients

2.3.3 General members

For a general beam element, the pertinent deformation are axial, major axis flexural, minor axis flexural, major axis shear, minor axis shear and torsional. The above sizing techniques can be extended to include all of these deformations and, although it is clear that axial deformation is a direct function of the cross-sectional area, the other deformations merit some discussions.

In general steel shape sections do not have section properties that are linear functions of area for all the available product series. However, for those properties of interest such as the major and minor axis moment of inertias and the major and minor axis shear areas a linear approximation can be found. Other remaining section properties, like the torsional constant, cannot be linearized so the energy density is not closely related to minimum volume sizing. Fortunately these deformations are often unimportant.

A simplified case is for rectangular sections. In these cross sections, in case of constant depth, the major and minor axis shear section properties are linear functions of the cross-sectional area and these functions approach zero as the cross-sectional area approaches zero. Therefore, a procedure similar to those above shows that a uniform energy density (element virtual work divided by element volume) will produce a minimum volume structure. The moment of inertia and torsional properties are non-linear functions of the area, therefore uniform energy density is not equal to the Lagrange multiplier and will not produce a minimum volume structure. However, these deformations are often unimportant in many structural systems.

In the design it is often sufficiently accurate to assume that all the properties are linear functions of cross-sectional area which intersect the origin as done for the major axis flexural bending. Whit such hypothesis a resizing algorithm can be easily found starting from the expression of the total virtual work v_i for a general beam element of length L:

$$v_i = \int_0^l \frac{n_i N_i}{EA_i}\, dx + \int_0^l \chi_t \frac{t_i T_i}{EA_i}\, dx + \int_0^l \frac{m_i M_i}{EI_i}\, dx \tag{39}$$

Where:

n_i = Axial force in the element due to the unit virtual load
N_i = Axial force in the element due to the real load applied
A_i = Element area
t_i = Shear force in the element due to the unit virtual load
T_i = Shear force in the element due to the real load applied
χ_i = Shear factor
m_i = Bending moment in the element due to the unit virtual load
M_i = Bending moment in the element due to the real load applied
I_i = Element inertia

The optimal element area can then be found with the following expression:

$$A_i = \frac{1}{\Delta_{req}} \sqrt{\frac{\nu_i}{L_i}} \sum_{j=1}^{n_{el}} \left(\sqrt{L_j \nu_j}\right) \tag{40}$$

A key part of the above discussion is that this method provides the minimum volume of material for statically determinate structures since the member force do not change as the member size change. This may seem to limit the usefulness of this method but a closer look shows that it can be applied in many cases through an iteration process.

Iteration process

The iteration process allows to deal with a redundant structure moving in each step the material from one member to another on the base of the new values of area and inertia assigned in each iteration to each member. If in each iteration the sections can be chosen with area and inertia values close to the previous one, then the process will converge in few steps.

Generally the first step, when from the arbitrary sections the optimal areas are found, reaches a configuration close to the final one and gives the more significant result in terms of importance of each single member compared to the whole structure, evaluating the total work done by all the element.

The following iteration steps are necessary because in choosing new section, being the structure redundant, the distribution of force in members change and therefore also the work done by the elements. The main point to be discussed, related to the iteration, is the same discussed for the flexural members: the optimization process gives as a cross-sectional area value and different values of inertia can be associated with.

The choice of the inertia has a relative strong influence on the distribution of stresses in the structure element during the iteration process and can be an important factor in ensuring convergence. From the point of view of the optimization the best choice seems to be the one with the section with the bigger inertia, in order to have with the same area better structural properties; on the other side a procedure like the one explained in section 2.3.1 allows to obtain a more homogeneous structure in terms of section properties. Both these two procedures has been evaluated in the next two example.

<u>Single module example</u>

The structure considered in this example is very similar to the structure analyzed in paragraph 2.3.1, but instead of a central pin that connects the two symmetric part of the structure a link fully fixed at both ends is introduced.

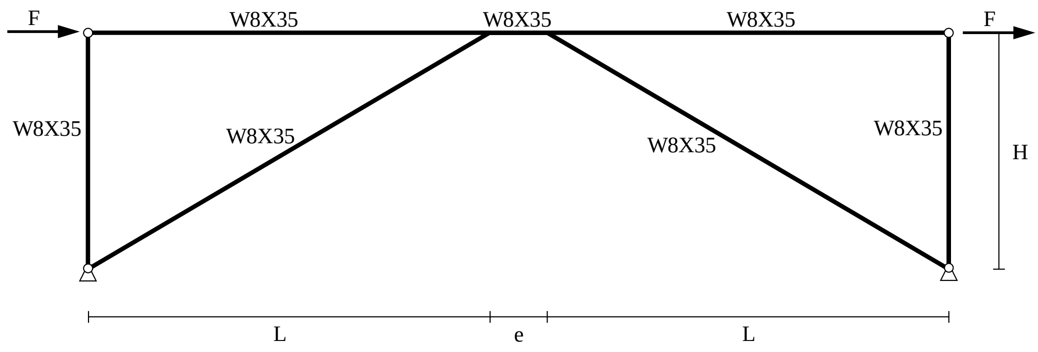


Figure 19: Geometry and section of the example structure

Data:

$L = 204\ in$

$e = 24\ in$

$H = 120\ in$

$F = 200\ kip$

The structure is redundant, two lateral point loads *F* of the same intensity are applied at the top of the brace, with height *H* and total length *2L + e,* and the sections are W-flange with an arbitrary area equal for all the elements.

In Figure 20a, b and c, is represented the stresses distribution under the real load, respectively axial, shear and flexural component.

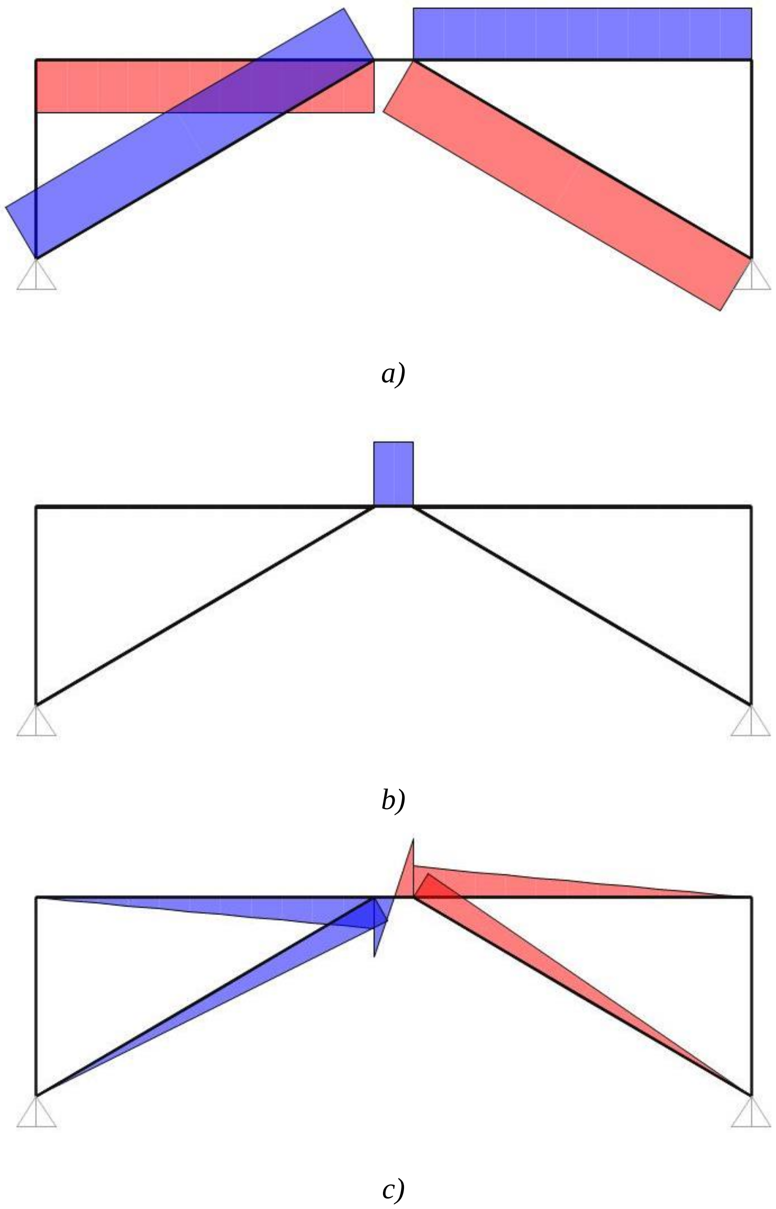


Figure 20: Axial a), shear b) and flexural stresses under real load

It is possible to notice that, being a symmetric load condition, the link isn't subjected to axial stresses and generally the axial behavior is governed by diagonals and beams while shear and bending moment behavior are concentrated into the link.

The deformation of the structure at the top, under the given loads, is *0.47 in* and the target deflection is set to *0.05 in*. Using equation (40) the optimal areas found are the ones shown in Table 4.

Table 4: Optimized areas result

	A_i [in^2]
A_1	1.3
A_2	42.2
A_3	80.1
A_4	111.7
A_5	1.3
A_6	92.6
A_7	92.6

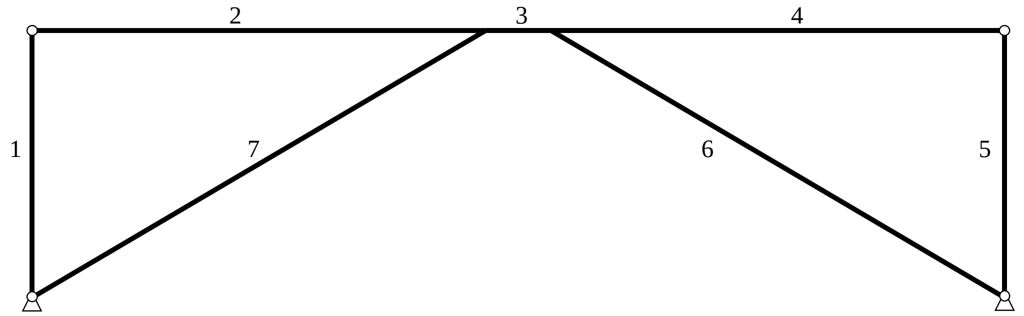


Figure 21: Members numbering of the example structure

The new areas are chosen from the list of the available sections and they are selected from a single series (W36). The sections in which the new area value is smaller than the initial one are kept the same; the values are smaller because the optimization process needn't to use these members in order to obtain the required displacement.

Table 5: Optimized result with commercial section

SECTIONS		**A_i** [in^2]		**I_i** [in^4]	
elem 1	W8X35	A1	10.3	I1	127
elem 2	W36X395	A2	116	I2	28500
elem 3	W36X282	A3	82.9	I3	19600
elem 4	W36X395	A4	116	I4	28500
elem 5	W8X35	A5	10.3	I5	127
elem 6	W36X330	A6	97	I6	23300
elem 7	W36X330	A7	97	I7	23300

The displacement with the new sections is *0.037 in*.

Being a redundant structure at least one iteration is necessary, in this way the value of the tip displacement should get closer to the required displacement (*0.05 in*) because the structure is already correctly sized with respect to the loading condition.

The displacement after the first iteration is *0.0497 in* with the following sections:

Table 6: Final optimized section after iterations

$A_{i,opt}$ [in^2]	
A_1	0.7
A_2	5.5
A_3	15.0
A_4	84.7
A_5	0.8
A_6	69.0
A_7	69.6

SECTIONS		$A_{i,final}$ [in^2]		$I_{i,final}$ [in^4]	
elem 1	W8X35	A_1	10.3	I_1	127
elem 2	W36X302	A_2	88.8	I_2	21100
elem 3	W36X135	A_3	39.7	I_3	7800
elem 4	W36X302	A_4	88.8	I_4	21100
elem 5	W8X35	A_5	10.3	I_5	127
elem 6	W36X247	A_6	72.5	I_6	16700
elem 7	W36X247	A_7	72.5	I_7	16700

The second iteration gives the same result therefore the convergence is reached in one iteration.

3

ECCENTRICALLY BRACED FRAMES

This section deals with eccentrically braced frame systems (EBFs), widely adopted in the design of steel structures as lateral resisting systems. The main property of combining stiffness with ductility is shown starting from a basic EBF configuration and its corresponding plastic mechanism. The main parameters that govern the EBF response under lateral loads are taken into account considering the link length and the energy dissipation mechanism. Consequently, results coming from experimental tests on EBF link beams are shown to point out clearly their actual behavior and help to outline their design philosophy.

3.1 Introduction

Eccentrically braced frames systems (EBFs) are a lateral load resisting systems for steel buildings that can be considered a hybrid system between a moment resisting frames (MRFs) and a concentrically braced frame (CBFs). EBFs are in effect an attempt to combine the individual advantages of CBFs minimizing their respective disadvantages. Several common EBF arrangements can be used in different situations and each of them requires a particular study in terms of behavior.

Although eccentric bracing has been long known for wind bracing, its application to seismic-resistant constructions is becoming very important in the last years. One of the first example of the excellent performance of EBFs under severe earthquake loading was demonstrated in a one-third-scale model frames at the University of California in 1977, studied by C. W. Roeder and Popov. Soon after this study, several major buildings were constructed incorporating EBFs as part of their lateral seismic-resisting systems.

Figure 22 Embarcadero 4 Building in San Francisco, CA

One of the first building using this “new” structural typology is the nineteen story Bank of America Building in San Diego and the forty-seven story Embarcadero 4

Building in San Francisco, Figure 22; both constructed around 1981. Since that time, several applications of this system have been adopted in practice in the following decades.

The distinguishing characteristic of an EBF is that at least one end of every brace is connected so that the brace force is transmitted through shear and bending of a short beam segment, called link, defined by an horizontal eccentricity between the intersection points of the two brace centerlines with the beam centerline (or between the intersection points of the brace and column centerlines with the beam centerline for links adjacent to columns). The link length is identified by the letter *e*.

Figure 23: Eccentrically braced frame example

The most important feature of EBFs for seismic-resistant design is the combination between their high stiffness with relevant ductility and energy dissipation capacity. The bracing members in EBFs provide a high elastic stiffness, similar to the one of CBFs, allowing the system to satisfy code drift requirements. Under a very severe earthquake loading condition, appropriately designed and detailed EBFs provide the ductility and energy dissipation characteristic of MRFs. The ductility of EBFs can be attributed to two factors. First, the inelastic activity under severe cyclic loading is concentrated exclusively to the links, which are designed and detailed to sustain large inelastic deformations without loss of strength. Secondly, braces are designed to avoid buckling, beyond the severity of lateral loads acting on the frame. The yielding of the links in EBFs allows to limit the maximum force transferred to the brace, acting as a fuse for bracing member loads, and the ultimate strength of the link can be accurately estimated. Thus, the prevention of brace buckling in EBFs allows stable hysteretic behavior under severe cyclic loading conditions.

The ductility and energy dissipation capacity of EBFs could be better understood by comparing the behavior of typical frames (MRFs and CBFs) under a cyclic load. Figure 24, Figure 25 and Figure 26 show typical lateral load versus displacement plots, experimentally obtained by Popov [3], for respectively an MRF, CBF, and an EBF system.

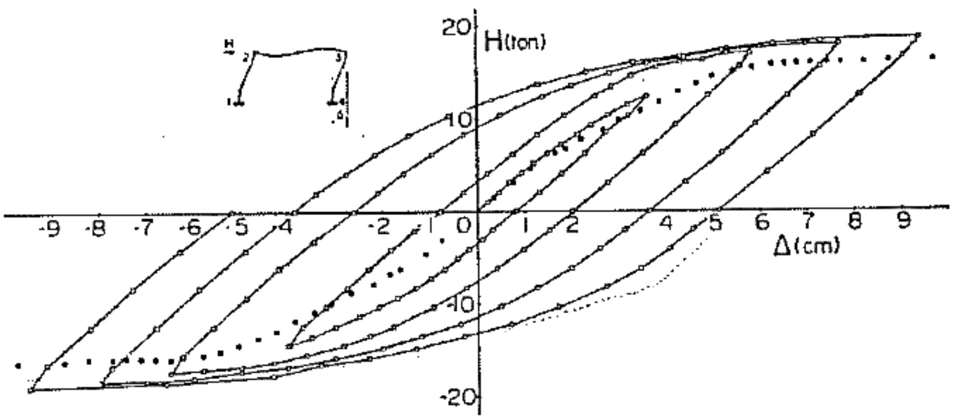


Figure 24: Moment resisting frame behavior (image is reproduced from [3])

The full and stable hysteretic loops in Figure 24 illustrate the MRFs ability to sustain large deformations without strength loss and are indicative of its high energy dissipation capacity. In contrast, the loops in Figure 25 are pinched and get worse as the number of loading increases, demonstrating the poor energy dissipation capacity of CBFs.

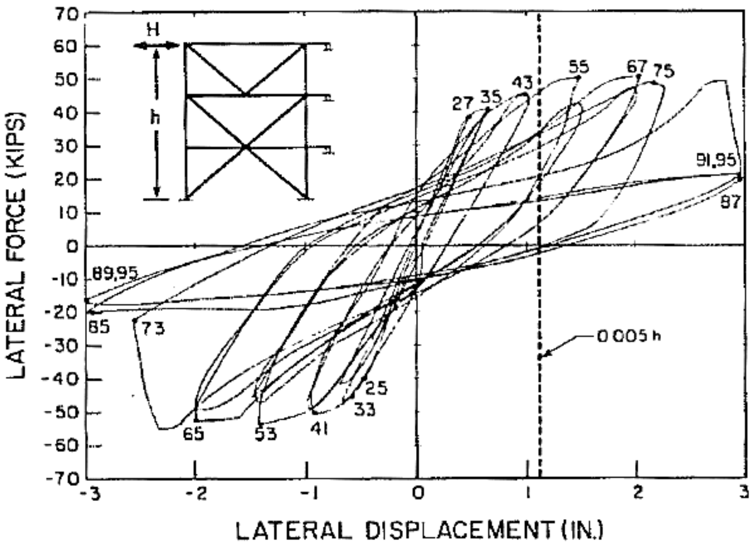


Figure 25: Concentrically resisting frame behavior (image is reproduced from [3])

This poor behavior is a result of the buckling of braces and their consequent rapid deterioration under cyclic load. Finally, Figure 26 illustrates the hysteretic behavior of a well-designed EBF. Because brace buckling is prevented and because the link can sustain large deformations without strength loss, full and stable hysteretic loops similar to those of MRF are obtained.

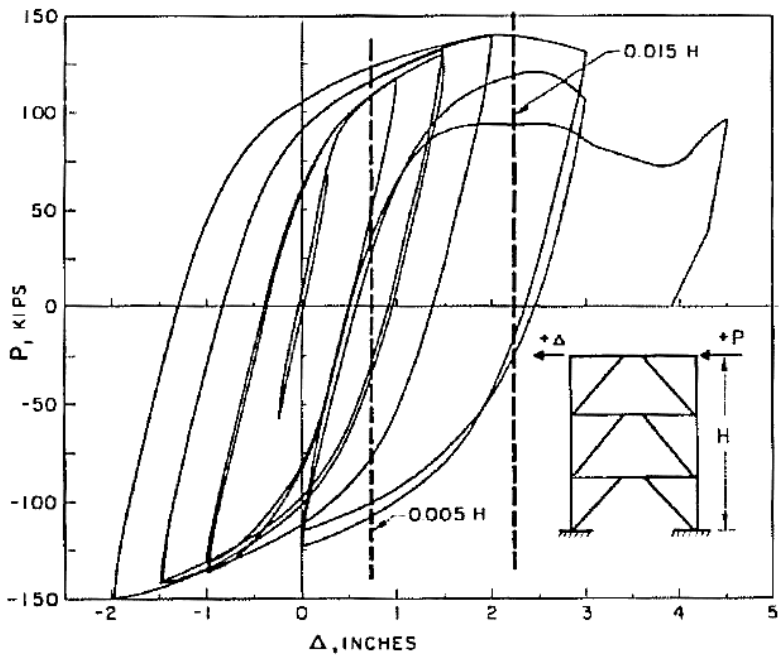


Figure 26: Eccentrically resisting frame behavior (image is reproduced from [3])

3.2 Eccentrically braced frame parameters

<u>*Stiffness and strength*</u>

It is very helpful to consider the variation of the elastic lateral stiffness of an EBF as a function of the link length *e*. This variation is illustrated in Figure 27 where two simple eccentric framing configurations are shown.

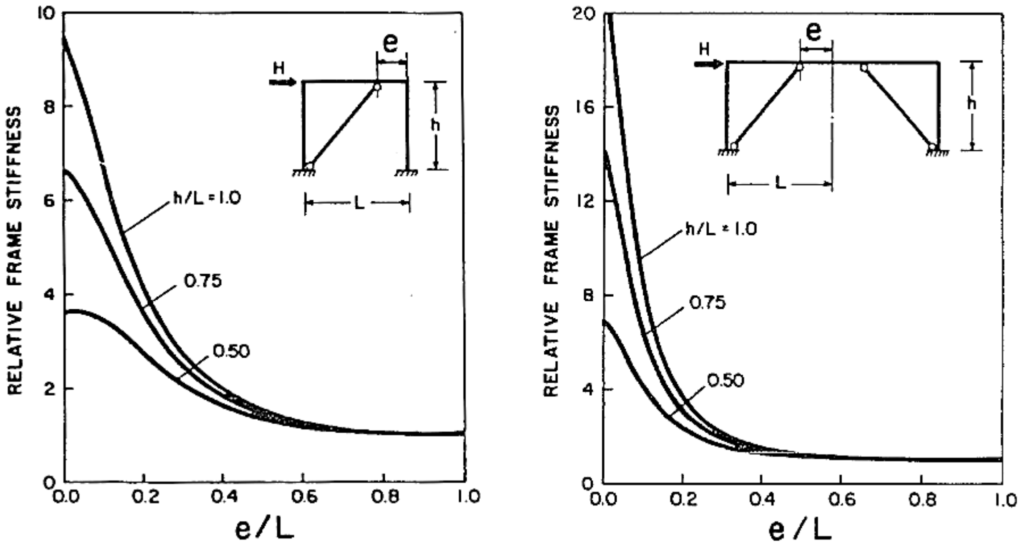


Figure 27: Relation stiffness – length e for different EBF configuration (image is reproduced from [3])

For $e = L$ one has a moment-resisting frame and the elastic stiffness is at a minimum. For *e/L > 0.5,* little stiffness is gained from the bracing and a rapid increase in stiffness occurs as the length of the link decreases. Maximum stiffness is exhibited when $e = 0$, corresponding to a concentrically braced frame. When $e = 0$, there is no link present to act as a fuse for brace member forces. It's clear that in order to gain maximum possible frame stiffness, the links must be kept short. However, links cannot be made too short because the inelastic deformation demand on the link would become excessive.

The link length also significantly affects the strength of an EBF under lateral load. Figure 27 illustrates the ultimate strength of a three-story EBF as a function of *e/h*, assuming elastic-perfectly plastic behavior. Frame capacity is normalized by the quantity $2M_p/h$ which represents the strength of an MRF. Frame strength rapidly increases in decreasing the link length, until the frame strength is limited by the fully plastic shear capacity of the links. This region of the frame behavior is represented by the horizontal lines in Figure 28. Clearly, maximum frame strength is achieved with short link lengths.

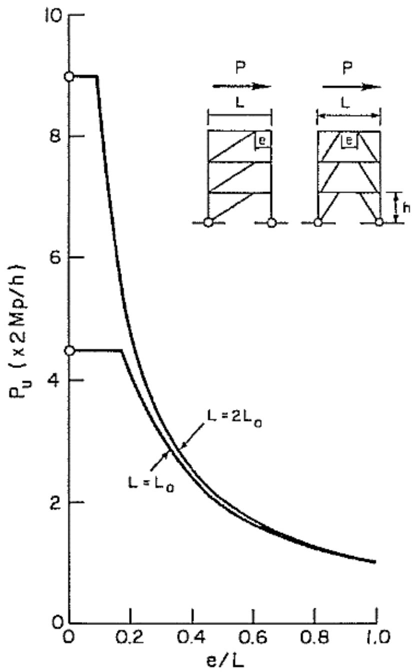


Figure 28: Relation e – strength (image is reproduced from [3])

It must be recognized that the effects of link length illustrated in the previous sections represent idealized situations for small frames, assuming constant member sizes as *e* is varied. The previous figures represent only a qualitative behavior as the link length varies: the actual behavior will depend on the real height of the building and its code drift limits.

<u>*Forces in links*</u>

Typical distributions of bending moment *M*, shear *V*, and axial force *P* in the beams and links of an EBF under lateral load are qualitatively illustrated in Figure 29, where two common eccentric framing arrangements are considered. From this figure, it is clear that the link is subject to high shear force along all its length and high bending moments at its ends.

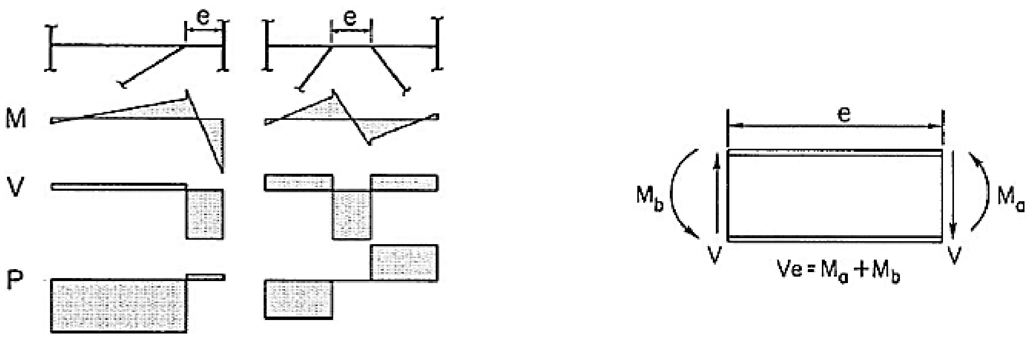


Figure 29: Forces distribution in link and beam of an EBF

If the links are kept very short, then under increasing lateral load on the frame the links will yield (activating plastic shear hinges) with relatively little yielding moments at its ends. On the other hand, if they are very long, the links will form conventional plastic moment hinges at the ends, with little or no shear yielding. As a result, short links are usually referred to shear links, and long links to moment links. The energy dissipation and ultimate failure mechanism for these two typologies of links differ substantially. Obviously, an intermediate behavior with both shear and moment yielding is exhibited for links with an intermediate length. As it will be discussed later, the short shear links provide for the best overall EBF behavior.

Forces acting on an isolated link and the relationship between shear and bending moment in the link based on static equilibrium are illustrated in Figure 29. In the case where the link end moments are of equal magnitude, then:

$$M_a = M_b = M \tag{41}$$

And the equation reduces to:

$$V_e = 2M \tag{42}$$

Considering the plastic moment in equation (43) and the plastic shear in equation (44) of a W-section, from equation (42) it is simple to derive the length of the link that is the theoretical dividing line between a link that yields in shear and a link that yields in bending moment:

$$\mathrm{M}_p = ZF_y \tag{43}$$

$$V_p = 0.6F_y t_w d \tag{44}$$

$$e = \frac{2M_p}{V_p} \tag{45}$$

Where:

Z = Plastic section modulus
F_y = Yield strength
t_w = Web thickness
d = Web depth
F_y = Yield stress

Thus, based on simply plastic theory, if $e < 2M_p/V_p$, the link shear will reach V_p before the end moments reach M_p, and the link will yield in shear. However, experiments showed that also strain hardening effects should be considered in understanding the link actual behavior.

As a result, in order to assure the behavior of links that yield in shear, it is recommended that the link length complies with the following equation:

$$e < 1.6\frac{M_p}{V_p} \tag{46}$$

3.3 Energy dissipation mechanism

In the design of a seismic resistant EBF, it is necessary to estimate the plastic demand of the link. This is accomplished through the use of energy dissipation mechanisms (collapse mechanisms), constructed by assuming rigid plastic behavior members. Mechanisms for an MRF and two different types of EBF are illustrated in Figure 30.

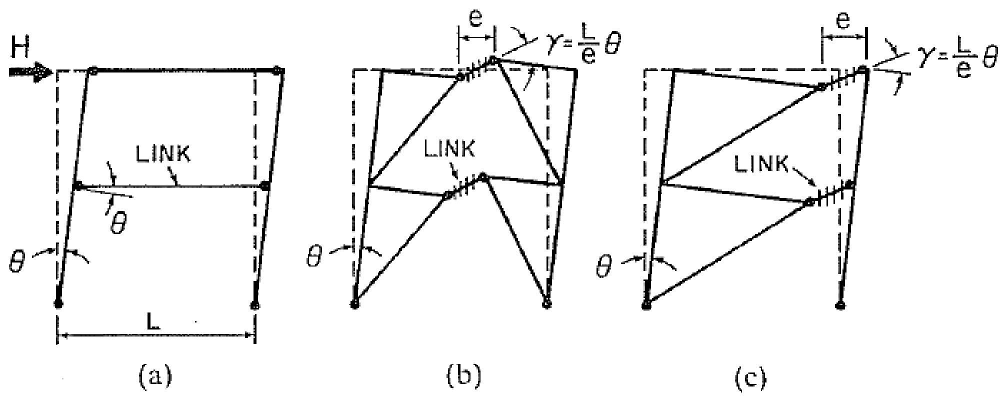


Figure 30: General teorical kinematism a) and EBFs mechanism belonging to different configurations b) and c) (image is reproduced from [3])

θ represents both the overall frame rotation at the base and the rotation demand at the plastic hinges of the beams for MRFs. However, for the EBFs the rotation demand on the links is larger than θ and it can be determined from the particular geometry of the system. In these two particular geometry situations it can be determined as follow:

$$\gamma = \frac{L}{e}\vartheta \tag{47}$$

In Figure 30 the links are cross-hatched to indicate that they yield in shear and form a shear hinge.

Figure 31 is a plot of link rotation demand versus *e/L* ratio for a particular type of EBF configuration.

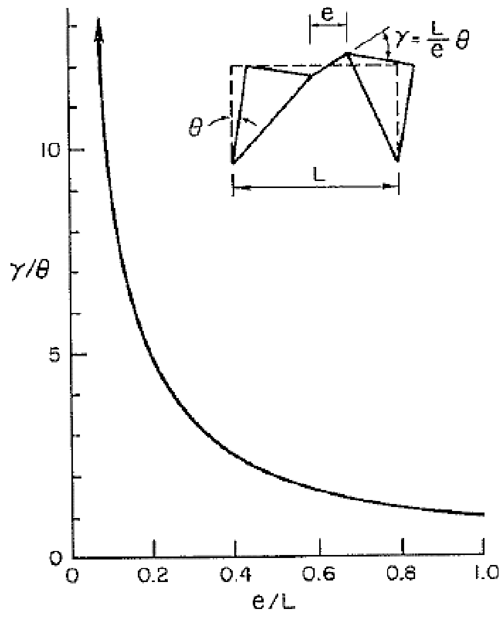


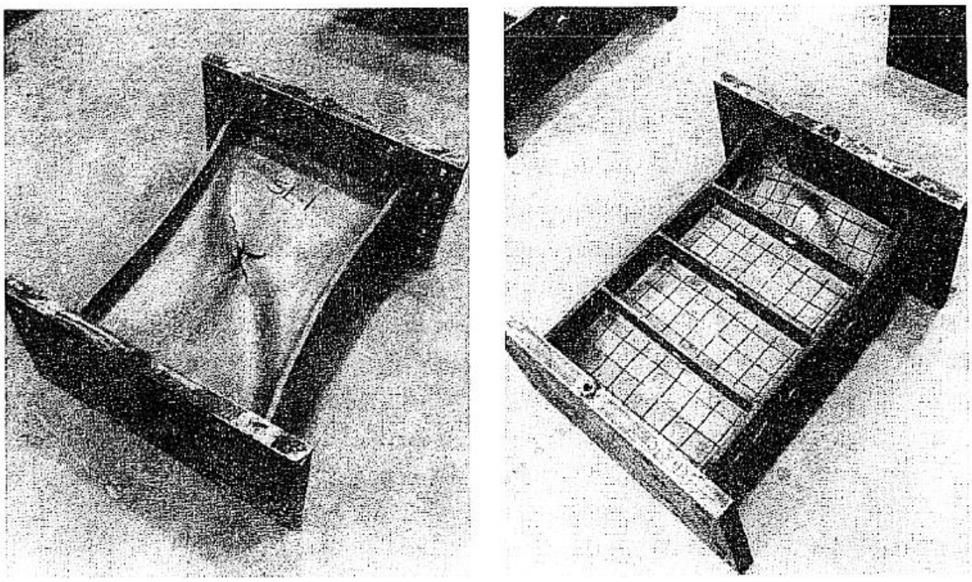
Figure 31 Relation rotation-link length (image is reproduced from [3])

This plot clearly illustrates that plastic rotation demand is larger in EBFs than in MRFs. The link rotation demand grows rapidly as link length decreases. These rotation demands can be met only by links that yield in shear, links that satisfy equation (46). Figure 31 also demonstrates that links should not be too short, or else the rotation demand becomes excessive, even for a shear link. The actual plastic rotation capacity of shear links has been well established experimentally.

<u>*Tests*</u>

In order to study the effect of inelastic web buckling in links and to better understand link behavior, some test were conducted in the past by Popov, University of California Berkeley [3].

In the first series, fifteen full size links were subjected to quasi-statically applied cycles of increasing relative end displacement. An example of link behavior from this first series is shown in Figure 32. Both specimens shown are *W18X40* sections, *28* inches length ($e = 1.11M_p/V_p$) of A36 steel.



a) b)

Figure 32: Deformations of link without stiffners a) and with stiffners b) (image is reproduced from [3])

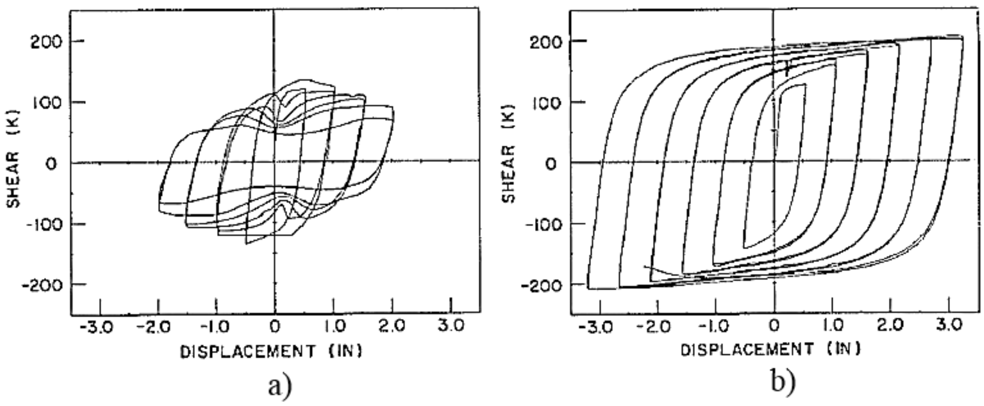


Figure 33: Hysteretic loops of link without stiffners a) and with stiffners b) (image is reproduced from [3])

The unstiffened specimen illustrated in Figure 32a experienced severe web buckling shortly after shear yield had occurred, causing deterioration of load-carrying capacity. The pinched hysteretic loops in Figure 33a indicate poor energy dissipation and ductility. The specimen provided with three pair of stiffeners, Figure 32b and Figure 33b showed a clear improvement in performance. The specimen achieved large inelastic rotations and the hysteretic loops remained full for a large number of severe loading cycles, indicating big energy dissipation capacity. For shear links, the plastic rotation γ can be closely estimated by the relative end displacement of the link divided by the link length. The elastic component of the relative end displacement for shear links is very small and can be neglected when computing γ. The stiffened specimen achieved relative end displacements of ±3 in, giving a plastic rotation capacity γ of about $\pm 0.10\ rad$. Other tests have confirmed that plastic rotation capacities of $\gamma = \pm 0.10\ rad$ can be achieved by well stiffened shear links. Note also that the stiffened specimen achieved an ultimate shear strength of approximately $210\ kips$. The nominal shear yield capacity of an *A36 W18X40* section is $V_p = 122 kips$. The actual shear capacity of this specimen, based on tests on the web was $122\ kips$. Thus, this specimen achieved an ultimate shear strength of about *1.9* times the nominal V_P or 1.7 times the actual V_P. Although this particular specimen experienced a rather unusual degree of strain hardening, it shows that the code-specified ultimate strength of a shear link (*1.5* times the nominal V_P for AISC) is not overly conservative and can be exceeded.

Some important observations from these series of tests are here summarized:

- Shear links can achieve larger plastic rotations and greater energy dissipation that moment links.
- Inelastic web buckling in shear link leads to significant loss in load-carrying capacity, plastic rotation capacity and energy dissipation. Web buckling can be avoided by reinforcing the web with stiffeners.
- A well-stiffened shear link can achieve plastic rotations up to $0.1\ rad$.
- Shear links suffer strain hardening, reaching ultimate shear strength on the order of 40-50% of the initial shear capacity.

<u>*Link length*</u>

If perfect plasticity and no M-V interaction are assumed, the theoretical passage between a shear link and a moment link is a length of $e = 2M_p/V_p$. The experimental results described previously indicate that the assumption of no M-V interaction is reasonable, but an assumption of a perfect plasticity is not. Shear links perform a significant strain hardening, generating ultimate shear forces on the order of *1.4* to *1.5* V_p to develop. One implication of strain hardening is that both shear and moment yielding will occur over a range of link lengths. For shear links, end moments substantially greater than M_P can develop. The large end moments combined with the dramatic strain gradients that occur in links lead to very large flange strains, which in turn can lead to failure at flange weld. In order to prevent excessive flange strains, Kasai and Popov suggest limiting link end moments to $1.2\ M_p$. Thus, from the link statics of Figure 29, if the end moments are limited to $1.2\ M_p$ and the link shear is assumed to achieve $1.5\ V_p$, the limiting link length is $e = 2(1.2M_p)/(1.5\ V_p) = 1.6M_p/V_p$. This is the limit also used in the AISC seismic provisions for steel structures [14].

When the link length is selected not greater than $1.6M_p/V_p$, shear yielding will dominate the inelastic response. If the link length is selected greater than $2.6M_p/V_p$, flexural yielding will dominate the inelastic response. For link lengths intermediate between these values, the inelastic response will occur through some combination of shear and flexural yielding. So the inelastic deformation capacity of links is generally greatest for shear yielding links and smallest for flexural yielding links. Based on experimental evidence, the link rotation angle is generally limited to *0.08rad* for shear yielding links and *0.02rad* for flexural yielding links (AISC).

EBF design philosophy

The material of this section is applicable to EBFs with shear links that meet the criteria of equation (46) and may not necessarily be appropriate for EBFs with longer links.

An early decision in the EBF design process is the choice of bracing arrangement. A guideline in choosing a bracing arrangement is generally to avoid brace-to-beam angles less than about *40* degrees. As the brace-to-beam angle becomes smaller, very large axial forces are generated in the beam segment adjoining the link, leading to potential strength and stability problems in this member.

The designer must also choose a link length at the preliminary stages: the use of shear links meeting the criteria of equation (46) is recommended. A useful guide at the early stages of design is to choose a link length on the order of *1.5* to *2* times the nominal beam depth. In general, it is possible to use longer links while still satisfying equation (46) by choosing heavier beam sections.

Strength and ductility are the two key design requirements for all seismic-resistant structure. In a well-designed EBF, the strength and ductility of the frame are directly related to the strength and ductility of the links. As a result of this relationship, the basic design philosophy for EBFs can be summarized as follows:

- Size the links to provide the required level of frame strength; detail the links to provide the required level of ductility.
- Design and detail the other frame members to be stronger than the links so that the strength and ductility of the links, and therefore the frame, can be fully developed.

With this approach, the links are designed for code or other earthquake forces. All other frame members, however, are not designed for code level forces, but rather for the forces generated by the fully yielded and strain-hardened links. These represent the maximum forces that can occur in these members regardless of earthquake magnitude. If this design philosophy is followed, the maximum strength and ductility of the EBFs will be achieved by assuring that yielding in the frame is restricted to the links.

Once the link sections have been selected, all other frame members are designed to remain essentially elastic under the forces generated by the fully yielded and strain-hardened links. This requires an estimate of the ultimate shear force and end moments that can be achieved by a link. The ultimate shear force Vu should be taken as at least:

$$V_u = 1.5V_P \tag{48}$$

A basic premise of EBF design is that braces must not buckle. Braces are therefore designed for the axial force generated by the ultimate link shear given by equation (*48*). As noted in the review of experimental results, ultimate link shear forces may sometimes exceed somewhat the value of 1.5 V_p due to overstrength of the web or due to the presence of a thick composite concrete deck. A conservative design of the brace is therefore appropriate. Note that the shear force in the beam segment outside the link also contributes to the axial force in the brace. Also, a portion of link end moment will be transferred to the brace, and this moment should be included in the brace strength and stability computations. As illustrated in Figure 29, the beam segment adjoining the link is subjected to a large axial force and a large bending moment and must be therefore be treated as a beam-column.

It is very important to understand the effects and the influence that each component or parameter of an EBF system has in the behavior of the entire system in terms of strength, stiffness and ductility.

Starting from the study of the basic geometry of an EBFs system is possible to play with its governing parameters in order to use this typology of brace frame inside more complex and particular structural systems of a building. It can be used inside a moment resisting frame or inside a lateral system, where its ductility behavior can play an important role if needed, as in case of a building subjected to high seismic forces. The particular and different behavior of the brace and of the link, in general terms, it’s independent by the particular geometry situation, so they have been used in the next chapters in a quite different application. In the next chapters, the basic EBFs properties have been applied in the lateral resisting frame of a high-rise building, trying to take advantage of the very high stiffness of the brace and of the ductility behavior of the link components.

4

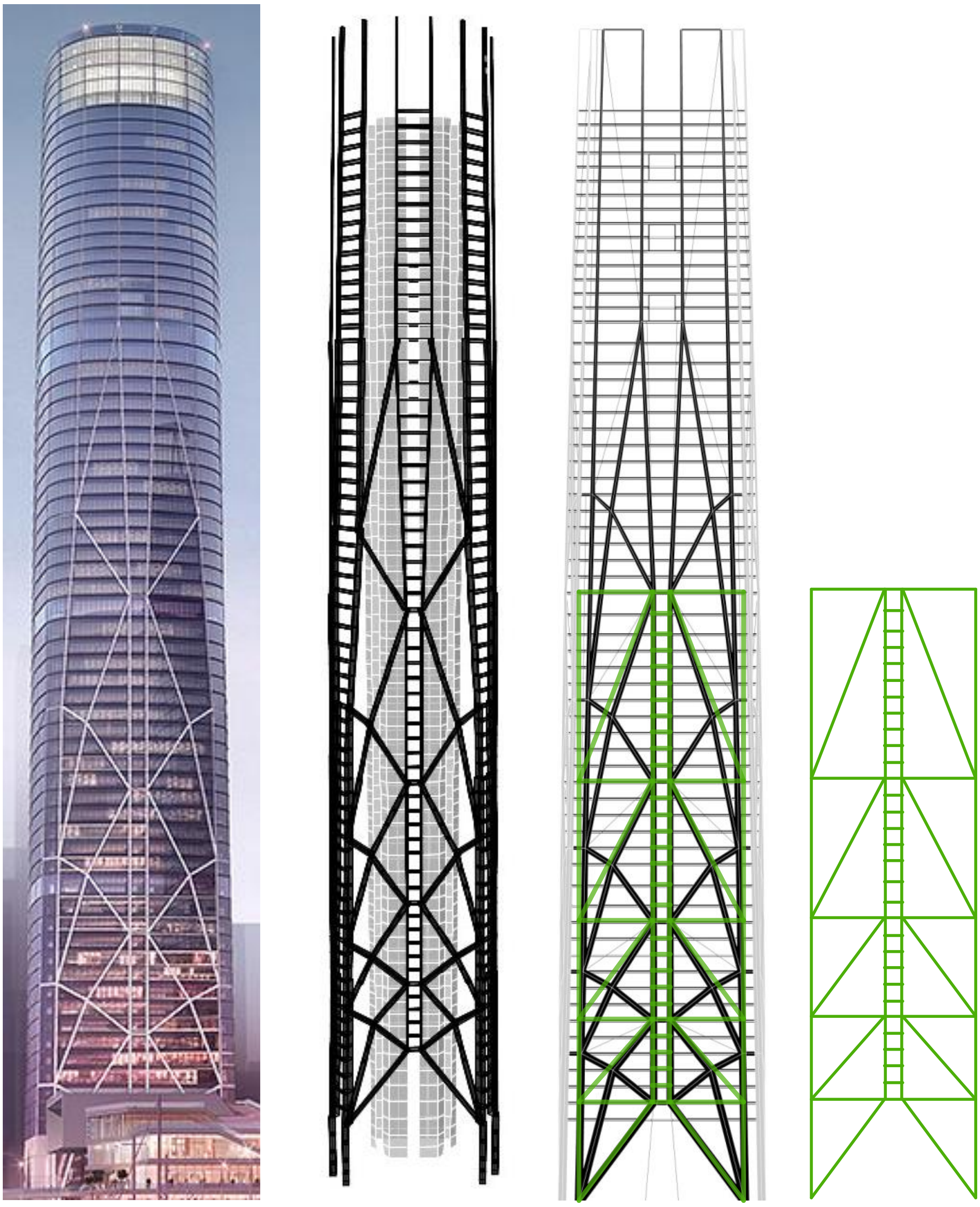
MODEL OPTIMIZATION

This chapter describes the model optimization run on a simple structure which, however, includes all the main features of a tied lateral system. The main goal is to combine in the most efficient way the elastic sizing optimization, using the energy based design method, with the dissipative behavior of a modified EBF system. Through a qualitative approach, it is possible to define the properties that columns, beams, diagonals and links have to exhibit, mainly related to elements stiffness, in order to give a positive contribution to the overall structural behavior. The structure obtained provides an adequate basis of comparison with the case study exposed in Chapter 5 and points out some guidelines to the design of tall buildings in which a lateral resisting frame is present.

4.1 Introduction

Since the aim is to combine the structural efficiency of the EBF system with the optimization theory showed for the simple structure, the plastic behavior has to be considered, but the optimization cannot be easily extended to the plastic field because of the material non-linearity. In fact, the strongest hypothesis under which the optimization is developed is that the section properties and the material properties remain constant through the steps. The iterative procedure explained for the redundant case is a way to overcome the problem of non-constant properties but at the same time shows how difficult is to deal with changing in the structural properties. Therefore, the plastic behavior is not implemented inside the optimization theory but it is confined in members that will not be optimized.

The analyzed structure is built in such a way that it is similar to the case study of Chapter 5 and therefore it will be possible to compare the two structures.



a) *b)* *c)* *d)*

Figure 34: Case study tower a), main structure 3D view b), lateral system plane view c) and simplified structure with series of EBF d)

The simplified structure model (Figure 34) is obtained combining many times a single EBF module showed in the previous chapter (e.g. Figure 23), with a central set of links connected at two columns that runs all along the height of the building. In this way, the real lateral system complexity of the case study is more simplified but the principal characteristics can still be analyzed. The lateral frame of the case study has to carry the horizontal force coming from seismic and a wind action and these loads are very different in the way in which they are applied to the structure. The seismic force is supposed to be a concentrated load that acts at each floor with highest value at the base, being that it is proportional to the weight of the floor, while the wind force is distributed along the structure with highest value at the top, being proportional to the height. The forces applied to the simplified structure are two symmetric concentrated loads acting at each floor, as it is shown in Figure 35 and a single module is considered in order to study the behavior of the EBF system with central columns and links.

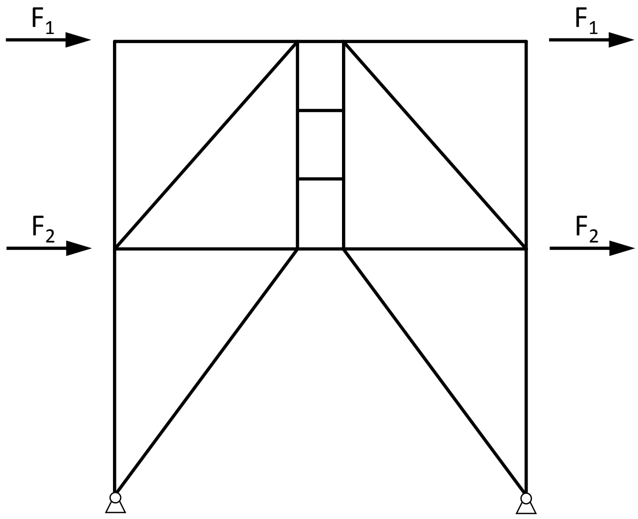


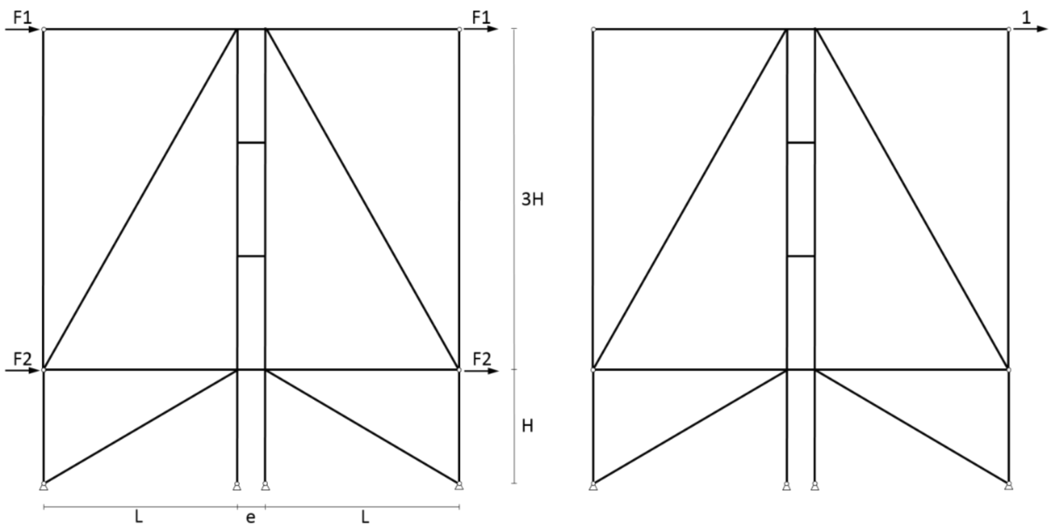
Figure 35: Single module simplified structure with concentrated symmetric loads applied

The lateral system has to be designed in order to dissipate energy with the plasticization of the links, while the others elements remain in the elastic field. First the elastic optimization is run for all the elements to have a preliminary sizing and to know the stress states in members, then the sections of links will be chosen in order to activate the plasticization.

4.2 Elastic optimization

The elastic optimization for the single EBF module follows the steps explained in Chapter 2: dimensions and loads are arbitrary and are not related to the magnitude of any real case, their values are set just to have a numerical example to be studied. Only the structural scheme configuration is related to the lateral system of the case study because the main aim is to recognize the structural role of each element and understand the principal features, distinguishing between the elastic brace and the plastic links. Once the principal structure features are understood, the optimization will be run on the case study, in the next chapter, with an in-house software called *OPTimizer*, developed by Skidmore, Owings & Merrill LLP.

The stress state is obtained with the structural software *SAP2000* and the result is found through an *Excel* spreadsheet in which the area optimization formula for general member is implemented. In Figure 36a it is shown the geometry of the simplified structure studied with the real force applied.



a) *b)*

Figure 36: Simplified structure geometry with real load applied a) and with unitary load applied at the right top b)

Data:

$L = 204\ in$

$e = 24\ in$

$H = 120\ in$

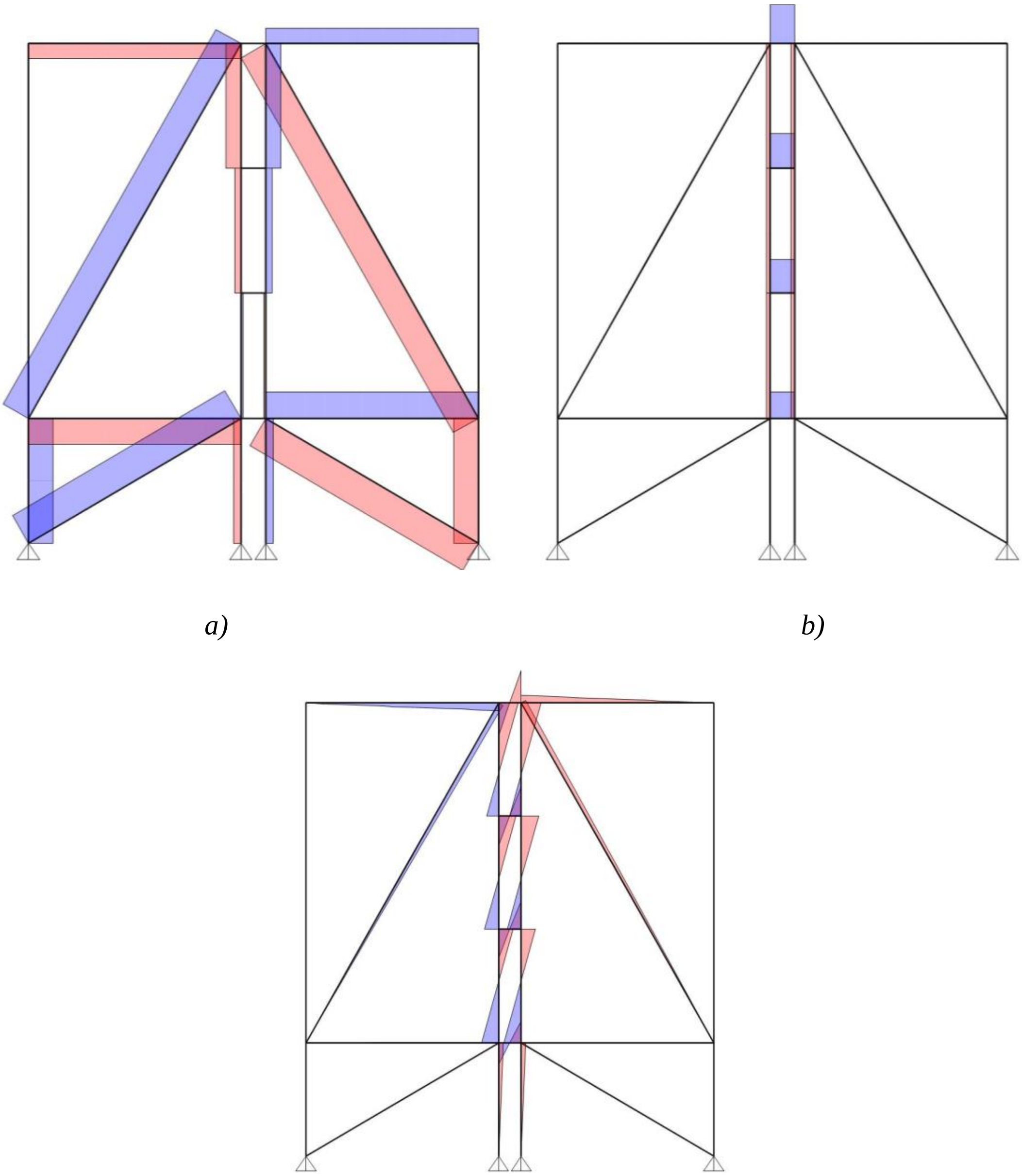
$F_1 = 170\ kip$

$F_2 = 130\ kip$

Initially all the structure elements have the same section (*W8X35*) and as usual, in order to optimize with respect to the tip displacement, a unit force is applied at the right top node (Figure 36b).

From the stress state under the real loads, shown in Figure 37, it is possible to get the main structural features of the brace system: on one side the behavior of the EBF, already explained, is maintained with high values of shear in the links and with the axial force mainly carried by the brace, on the other side the central elements come out to be very important with the two central columns that connect the links all along the structure. In fact the shear force inside the link attached to the brace (main link) is transferred to the other links above (secondary links) as axial load through the columns. It can be easily noticed that in every point in which a link is attached the amount of the axial load inside the column decrease with a discontinuity reaching smallest value towards the above EBF. Therefore the amount of shear force flowing from the links is controlled by the axial stiffness of the columns.

Another possible mechanism is that the shear force coming from the main link flows into the diagonals (as axial force and shear force) without going into the columns and without loading the secondary links. In this case the whole behavior of the structure would be controlled only by the deformation of the main link, while the two central columns and the secondary links would be not contributing to the behavior. Since the structure is redundant, in the iteration steps this effect has to be taken into account because, when the new values of area and inertia are chosen for each section, the stiffness changes and therefore also the force distribution changes.



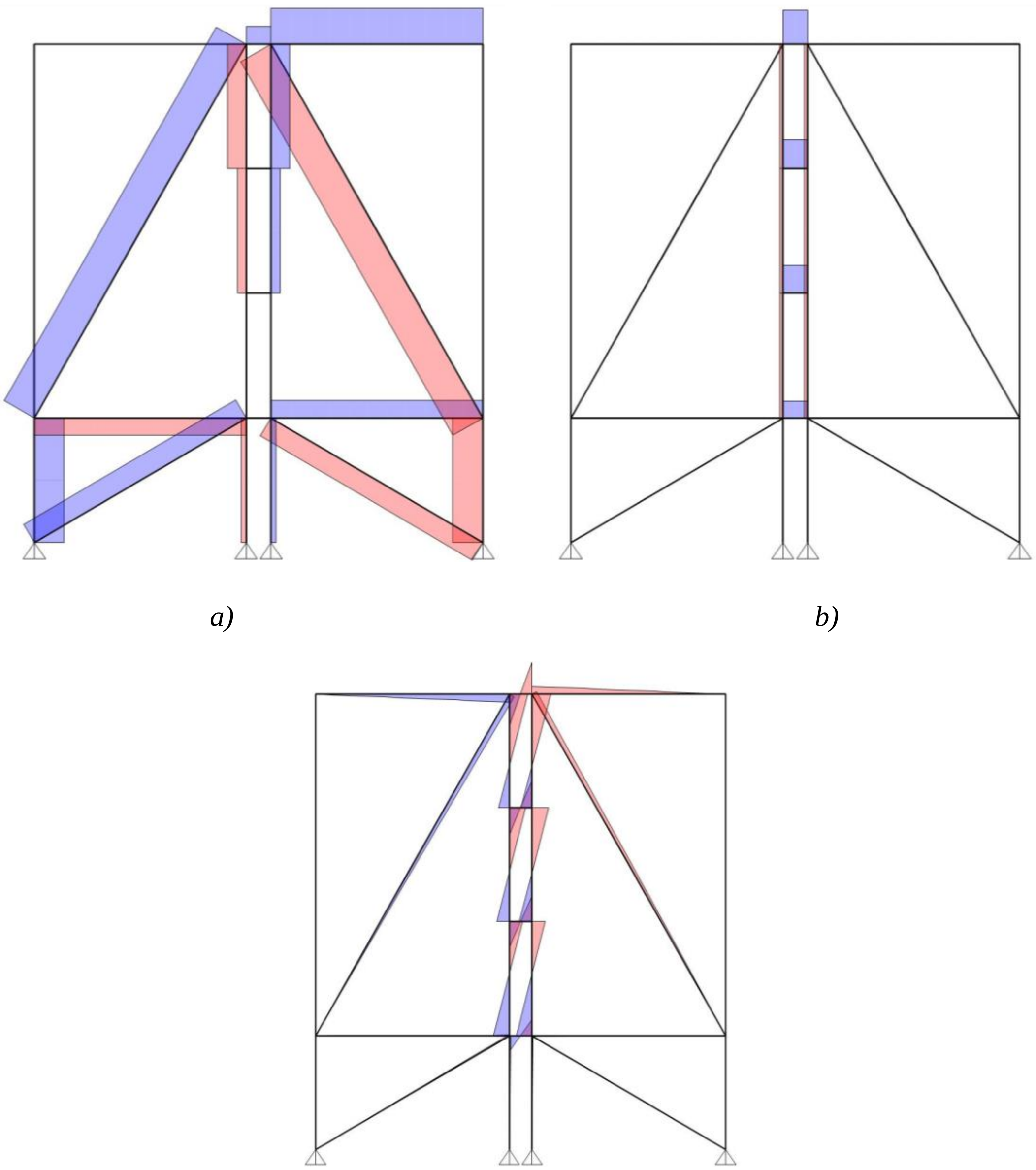
a) *b)*

c)

Figure 37: Axial a), shear b) and flexural c) stress distribution under real load

Another parameter that has to be considered is the flexural stiffness of the columns. If the secondary links are involved with high shear force levels, also a relevant bending moment will affect the columns behavior with a high flexural deformation related. Both flexural and axial columns stiffness are then the main keys to design correctly the brace.

Appling the unitary load, the stress distribution (Figure 38) is quite the same of the real load, with few differences related to the non-symmetric load case.



a) b)

c)

Figure 38: Axial a), shear b) and flexural c) stress distribution under unit load

Starting from arbitrary areas, $A_{i,ini}$, of the structure (*W8X35* for all sections, tip displacement *1.94 in*), set the required displacement to *0.5 in*, the optimization is run using the equation (49) for general member with equation (50) calculated with axial, shear and flexural component.

$$A_i = \frac{1}{\Delta_{req}} \sqrt{\frac{\nu_i}{L_i}} \sum_{j=1}^{n_{el}} \left(\sqrt{L_j \nu_j} \right) \tag{49}$$

$$\nu_i = \left[\int_0^l \frac{n_i N_i}{EA_i}\, dx + \int_0^l \chi_t \frac{t_i T_i}{GA_i}\, dx + \int_0^l \frac{m_i M_i}{EI_i}\, dx \right] A_i \qquad (50)$$

In order to distinguish each structure element the numbering showed in Figure 39 is followed: the main links are number *24* and *3* while the two secondary links are number *17* and *10*.

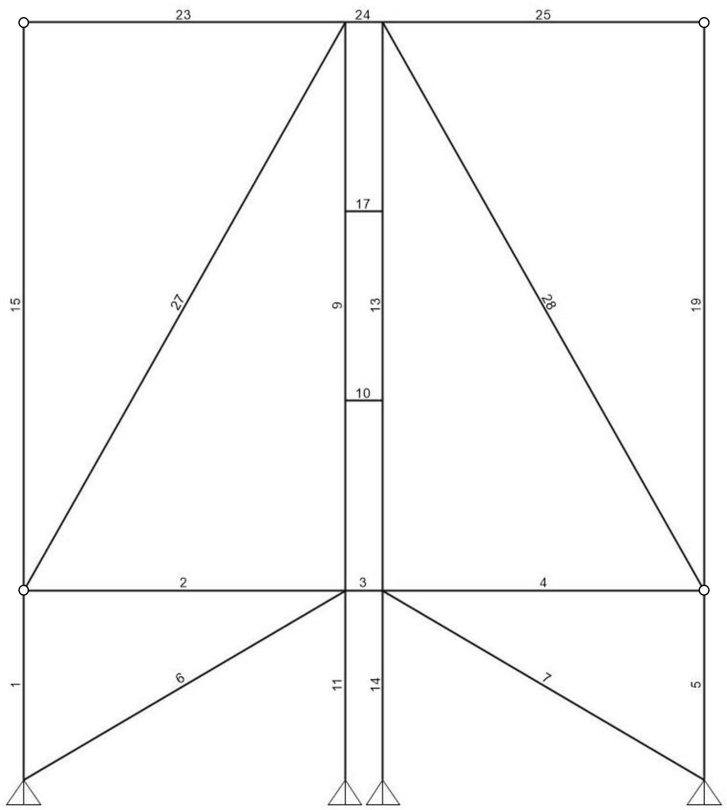


Figure 39: Studied module geometry

Running the optimization the new area values obtained, $A_{i,opt}$, are:

Table 7: Result of the first optimization with no iterations

$A_{i,\,ini}$ [in^2]		$A_{i,\,opt}$ [in^2]		SECTIONS		$A_{i,\,final}$ [in^2]		$I_{i,\,final}$ [in^4]	
A_1	10.3	A_1	41.8	elem 1	W36 × 150	A_1	44.2	I_1	9040
A_2	10.3	A_2	32.6	elem 2	W30 × 116	A_2	34.2	I_2	4930
A_3	10.3	A_3	23.5	elem 3	W18 × 86	A_3	25.3	I_3	1530
A_4	10.3	A_4	32.6	elem 4	W30 × 116	A_4	34.2	I_4	4930
A_5	10.3	A_5	42.0	elem 5	W36 × 150	A_5	44.2	I_5	9040
A_6	10.3	A_6	39.5	elem 6	W36 × 135	A_6	39.7	I_6	7800
A_7	10.3	A_7	39.5	elem 7	W36 × 135	A_7	39.7	I_7	7800
A_9	10.3	A_9	25.8	elem 9	W18 × 97	A_9	28.5	I_9	1750
A_{10}	10.3	A_{10}	21.7	elem 10	W18 × 76	A_{10}	22.3	I_{10}	1330
A_{11}	10.3	A_{11}	9.0	elem 11	W18 × 35	A_{11}	10.3	I_{11}	510
A_{13}	10.3	A_{13}	26.1	elem 13	W18 × 97	A_{13}	28.5	I_{13}	1750
A_{14}	10.3	A_{14}	9.0	elem 14	W18 × 35	A_{14}	10.3	I_{14}	510
A_{15}	10.3	A_{15}	0.2	elem 15	W8 × 35	A_{15}	10.3	I_{15}	127
A_{17}	10.3	A_{17}	22.9	elem 17	W18 × 86	A_{17}	25.3	I_{17}	1530
A_{19}	10.3	A_{19}	0.2	elem 19	W8 × 35	A_{19}	10.3	I_{19}	127
A_{23}	10.3	A_{23}	0.4	elem 23	W30 × 124	A_{23}	36.5	I_{23}	5360
A_{24}	10.3	A_{24}	25.9	elem 24	W18 × 97	A_{24}	28.5	I_{24}	1750
A_{25}	10.3	A_{25}	36.0	elem 25	W30 × 124	A_{25}	36.5	I_{25}	5360
A_{27}	10.3	A_{27}	48.3	elem 27	W36 × 170	A_{27}	50.1	I_{27}	10500
A_{28}	10.3	A_{28}	48.6	elem 28	W36 × 170	A_{28}	50.1	I_{28}	10500

The sections are chosen ($A_{i,final}$) in such a way that the structure is symmetric and when the area required is smaller than the arbitrary one the section doesn't change: the displacement at the top with these new sections is *0.45 in*. An iterative procedure is needed being the structure redundant and after two iteration the displacement is exactly *0.5 in*.

4.2.1 Identification of two different systems

In the iterations process, the optimization tends to reduce the area given to the columns and to the secondary links, increasing the main links area, as it's clear from Figure 40. In this graph, the iteration process is shown for some structure elements using commercial area values.

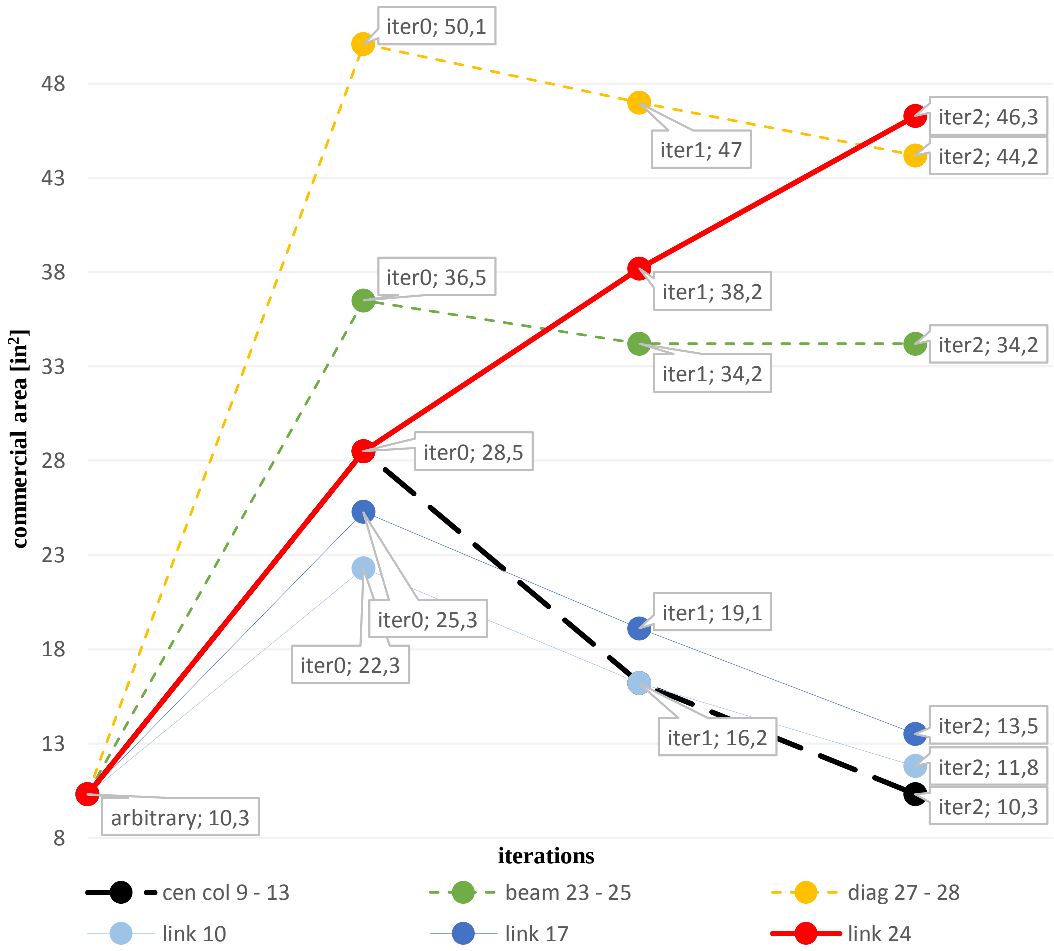


Figure 40: Changing of areas during iteration

It can be seen that the main link (element *link 24*) and the columns (elements *col* 9 – 13) have a completely opposite behavior: after the first iteration, the main link area increases because the optimization gives only to this link and to the brace the necessary stiffness to resist the lateral load. As a consequence, to obtain the required displacement, the set of secondary links (elements *link 10* and *link 17*) and the central columns are not relevant anymore and therefore the main link area is highly increased in order to transfer all the force coming from the beams (elements *beam 23 – 25*) only to the diagonal members (elements *diag 27 – 28*). The reference node where beam, diagonal and column converge into the link end is clarified in Figure 41.

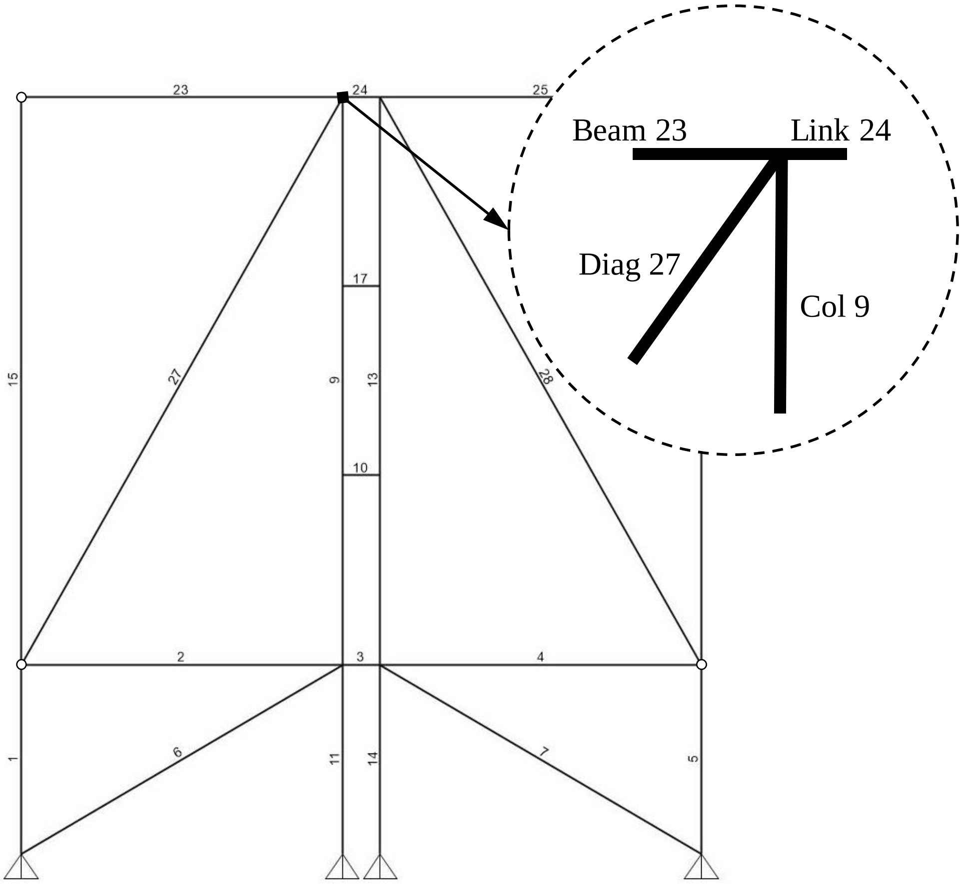
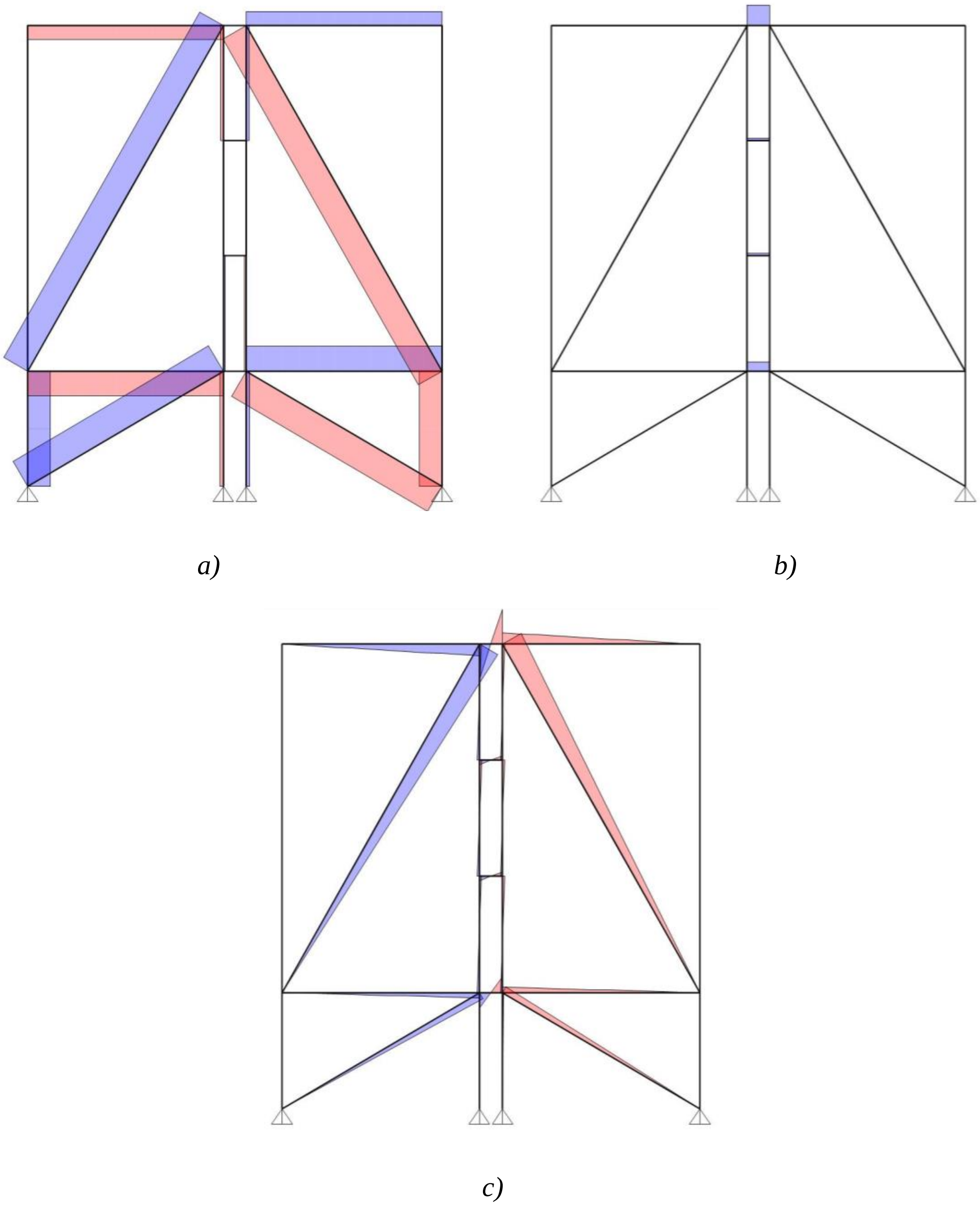


Figure 41: Reference node zoom whit main link, column, diagonal and beam

It must be taken into account that the value of area required in the link by the optimization, being the link mainly a flexural element, it's strongly correlated to the inertia of the member. In choosing the link commercial section, a shape with the biggest inertia related to the smallest area would be the best choice but during the iteration, to help the optimization to converge quickly it is better to choose a section in the same series of the previous one to maintain the same area-inertia relation.

However in a member sizing like the one achieved, as Figure 42b clarifies, the links are subjected to very different shear and flexural actions with the main link that is highly stressed and the secondary links that can't participate to the flexural behavior of the whole structure (Figure 42c). In addition, for the same reason (Figure 42a), the axial force inside the two central columns is very low.



a) *b)*

c)

Figure 42: Axial a), shear b) and flexural c) stress distribution under the real load after the second iteration

This fact suggests that the optimization process is finding a solution to match the required displacement using only the stiffness of the brace without involving the two central columns and consequently the secondary links. Being the links the structural elements that will be designed to undergo the plasticization, with these values of force acting on them the plasticization would be impossible to reach.

Moreover, in this way the central system of columns and secondary links is completely useless.

From the stresses distribution and considering what is just said, it can be noticed that the structure can be divided in two part as shown in Figure 43: a brace system (blue line) that is the main structure resisting to the lateral loads and a central system (red line) in which the shear force has to flow from the main link to the secondary links throughout the columns.

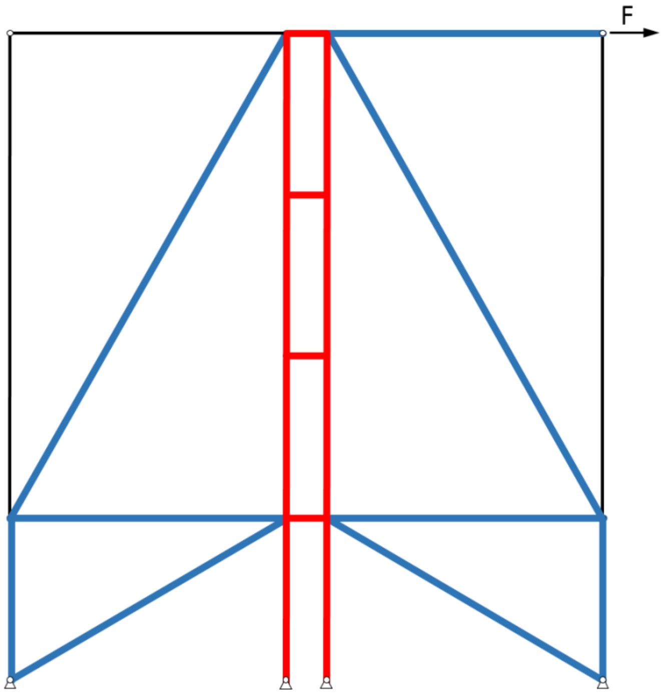


Figure 43: Identification of two different part of the structure

From this division, the brace, being the main component of the structure, will be elastically optimized while the remaining part will be the system in which the plastic behavior will be concentrated, with the set of link forming the plastic hinge; moreover the aim is to involve in the most homogeneous manner all the links of the structure to have a good plastic behavior.

In order to succeed in this path the central columns and the secondary links don't need to be iterated in the optimization, therefore the idea is to constrain these elements sections in order to keep a constant value of area or inertia inside the optimization process: in this way the iterative procedure for the optimization can be run for the whole structure with some elements not involved.

The constrain procedure, from the theoretical point of view as presented in [15], means to calculate separately the work done by the constrained elements, moving it from the total amount of work done by the structure to the right side as equation (51) shows. A_i are the areas to be found, and the A_j are constant area values to be held.

$$\sum_{i=1}^{n}\left(\frac{P_i p_i L_i}{EA_i}\right) = \Delta - \sum_{j=1}^{m}\left(\frac{P_j p_j L_j}{EA_j}\right) \qquad (51)$$

Then, starting from this expression of the total virtual work, the formulation is the same explained in chapter 2 where the single element area A_i are obtained.

In running the optimization the section elements belonging to the central system will be constrained to a certain value and will be sized, outside of the optimization process, in order to have a homogenous links plastic behavior, while the brace will be fully sized through iterations.

4.3 Plastic link design

Before designing the central system of columns and links with the definition of the plastic hinge, the shear force acting on the links has to be known (Figure 44) and therefore a preliminary optimized sizing is needed to have the proportions between the sections and the stresses distributions. Using the result of section 4.2 (Table 7) it can be noticed that when the first optimization was run, starting from an arbitrary configuration with all section with the same area, *links 24*, *17* and *10* had more or less the same section (Figure 45) and the tip displacement was *0.44 in*, satisfying the required one of *0.5 in*.

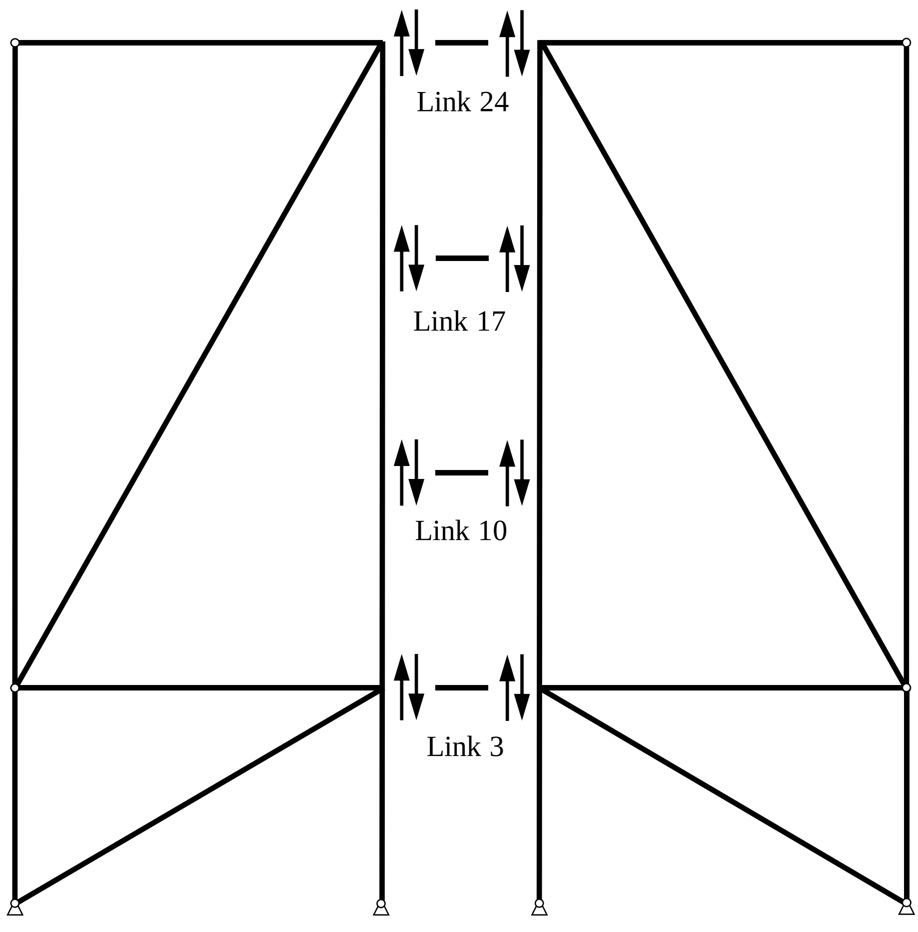


Figure 44: Shear force on main links and secondary links

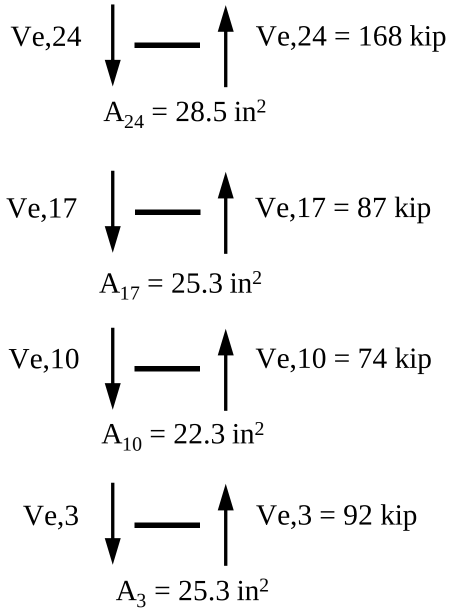


Figure 45: Shear and area values of links after the first optimization

Further iterations are not performed because with the first iteration the section of *link 24* will increase while, at the same time, sections of *links 17* and *10* will decrease, as is clear from Figure 40, with a consequently redistribution of shear force among them.

This fact allows to have the shear forces coming from a structure in which the main link is more loaded with respect to the secondary link, but at the same time the secondary links are involved with a relevant force value that easily allows the plasticization. If the full optimization with all the iterations had been considered, the shear on the main link would be of *241 kip* while on the secondary links the shear would be of *30 kip*, hence the plastic behavior wouldn't be so homogeneous. Therefore, first of all the sections link are sized, then further iterations for the elastic brace could be performed with the new link area values constrained inside the process.

After that the link sections are chosen to satisfy the plastic behavior, since it would be very cumbersome running the optimization with some constrained area values by hand as till now has been done in the examples proposed, the iterations will not be performed because an optimization software is necessary. In Chapter 5 the optimization with constrained area values will be run with the optimization software in which is implemented the same formula of equation (51).

As found out in section 3, the best plastic mechanism associated with an EBF structure is to have a yielding in shear inside the link because it allows bigger rotations then the flexural yielding; thus, before running the optimization, it is useful to understand how to define the plastic hinges inside the structure. The choice of the yielding in shear is also justified from the stress state acting on the link. In Figure 46 it can be seen that the shear is constant and the moment is linear: therefore, in order to define a link that yields in shear, a single plastic hinge will be defined in the middle, while on the other side a flexural link, having a linear distribution of bending moment, will lead to the definition of two plastic hinges at each side of the link

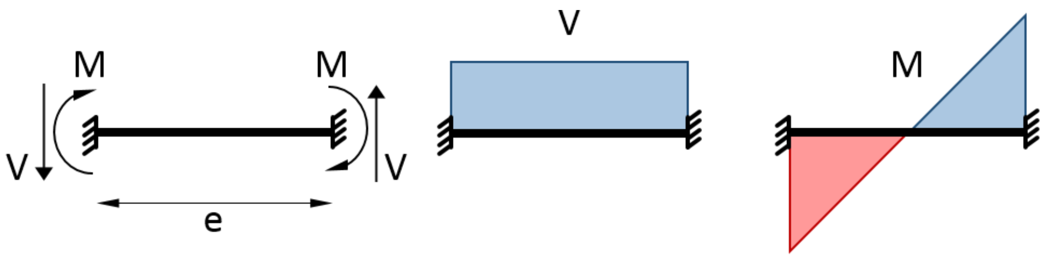


Figure 46: Shear and bending stresses acting on a link

In conclusion, a shear link has better structural behavior properties and it is simple to define, managing only one plastic hinge for each link.

The plastic hinge definition is set in *SAP2000* in the *section property* menu and a specific hinge is defined for each link section. Every hinge type is *deformation controlled* and the plastic behavior parameters are define with respect to the shear force *V* (Figure 47). The program allows the user to insert manually the yield force and the yield displacement that are section-dependent properties and lets the user also customize the constitutive law with the possibility to define three more point, corresponding to the acceptance criteria for the serviceability limit state.

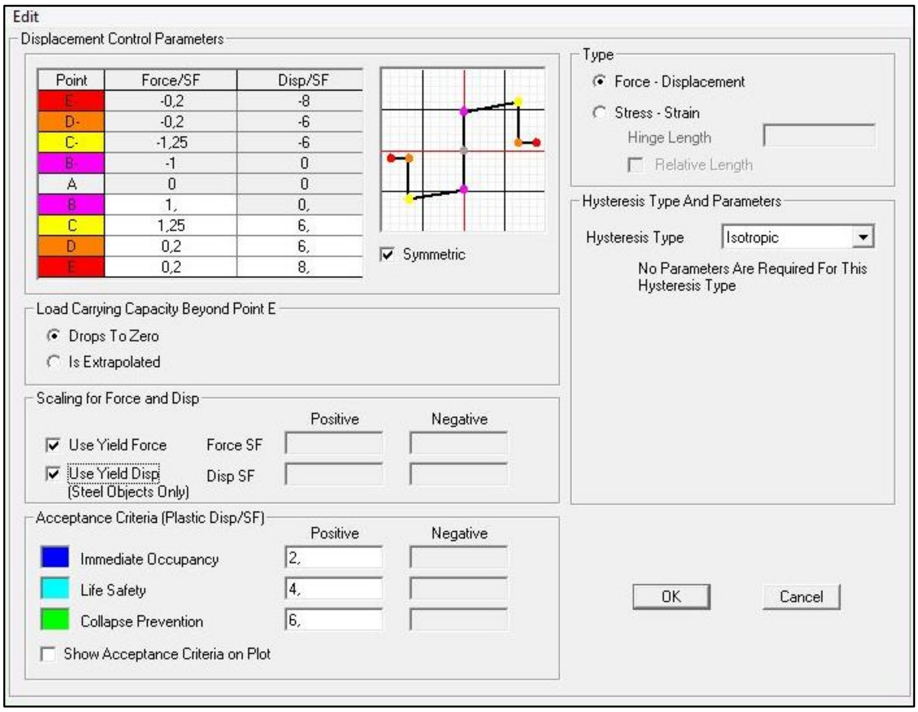


Figure 47: Window of SAP2000 for the definition of the plastic hinge behavior

In this case the plasticization of the link has been controlled through the value of the yield force that is the plastic shear defined in equation (53) while the various steps after the reaching of the yielding point are controlled by the link deformation also because the hinge behavior is perfectly plastic.

In the optimization process the shear force at the end of the link will be taken and set as the reference force under which the link has to be already in the plastic phase. On one hand the yield force has to be smaller than this reference shear value but on the other hand it has to be big enough to avoid the hinge collapse. For this reason, in this example, the following rule has to be satisfied:

$$V_p < V_e < V_c \tag{52}$$

Where:

V_e = Shear force at the link ends

V_p = Plastic shear

$V_c = 1.25V_p$ = Collapse shear

The collapse shear is higher than the plastic shear even if the plastic hinge is defined as elasto-perfectly plastic because in steel structure a certain amount of hardening is always present: the elasto-perfectly plasticity is a choice used in this design phase to simplify calculation and modelization of the plastic behavior. The plastic shear is a section property and depends on the material yield stress, it can be expressed through equation (53):

$$V_p = 0.6\left(\left(d - 2t_f\right)t_w\right) f_y \tag{53}$$

Where:

d = Section depth

t_f = Flange thickness

t_w = Web thickness

In order to have a shear link, as shown in chapter 3, the link has to be "short", this means that the length *e* has to be:

$$e \leq 1.6\frac{M_p}{V_p} \tag{54}$$

With

M_p = Plastic moment

The length of the links inside the EBFs are *24 in* and then the ratio between the plastic moment and the plastic shear of equation (54) has to be smaller than that value. The choice of the sections, according to this constrain, leads to have the values in Table 8

Table 8: Link section choice for shear yielding

LINK SECTION	A_{sec} [in^2]	A_w [in^2]	Z [in^3]	M_p [kip-in]	V_p [kip]	$\frac{1.6\,M_p}{V_p}$	**length e** [in]	**yielding**
W8 × 40	11.7	2.6	40	1990	77	41	24	shear yielding
W8 × 28	8.3	2.0	27	1360	61	36	24	shear yielding
W8 × 40	11.7	2.6	40	1990	77	41	24	shear yielding
W10 × 77	22.6	4.7	98	4880	141	55	24	shear yielding

The shear link acting on each link follows the limitation of equation (52).

Table 9: Shear link values

LINK SECTION	**V_p** [kip]	**V_e** [kip]	**V_c** [kip]
W8 × 40	77	92	96
W8 × 28	61	74	76
W8 × 40	77	87	96
W10 × 77	141	168	176

In conclusion each link section is chosen in such a way that the geometrical properties allow the plasticization under the shear force acting on the link and at the same time they guarantee the shear yielding behavior. In changing the links sections, being the structure redundant, also the force will change and in some case the plastic shear could result bigger than the new design shear force, as could happen for secondary links. Although this fact, plastic hinges in those links will be activated because after the main link plasticizes the exceeding shear will flow into the column towards secondary links increasing the shear force acting on them. In any case a check is needed to be sure that all the links undergo the plasticization.

4.4 Plastic structural behavior

Studying the activation of plastic hinges inside links means that the material non-linearity needs to be taken into account. In order to obtain an accurate and specific prediction of the structural response, it is necessary to have an analysis tool that allows getting the non-linear behavior with its evolution in time. As a first step, the kinematic of the structure can be considered to relate the deformation of the overall structure with the local deformation of the single link. This can be done in an easy way considering the single EBF module assuming that the same relations found with respect to this model hold still for the simplified structure module studied. Being the link the element in which the plasticization occurs, the definition of the plastic hinge will govern the behavior of the structure in the non-linear static analysis. It has to be noticed that the optimization tool used until now has the hypothesis of linear material behavior and then the study of an alternative way to size the structure elements, characterized by the non-linearity, has to be developed.

Considering the kinematic of the structure under a lateral force, it is simple to verify that the structure deformation in Figure 48 is governed by the relation between the base rotation, the tip displacement and the height of the module in equation (55).

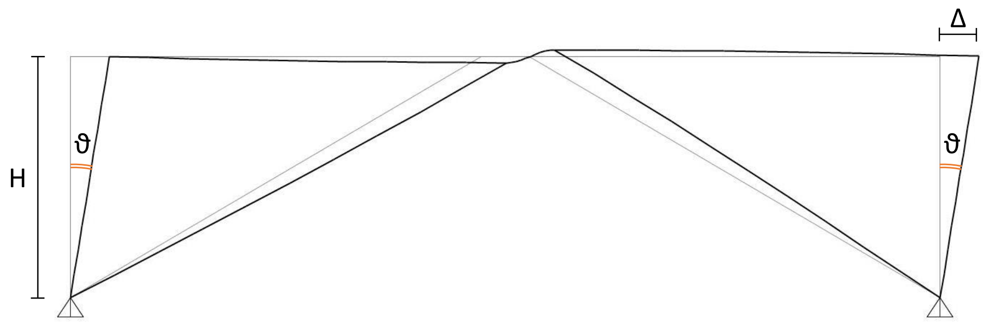


Figure 48: Standard EBF deformation

$$\Delta = H\vartheta \tag{55}$$

Considering the link deformation, equation (56) holds and is represented in Figure 49.

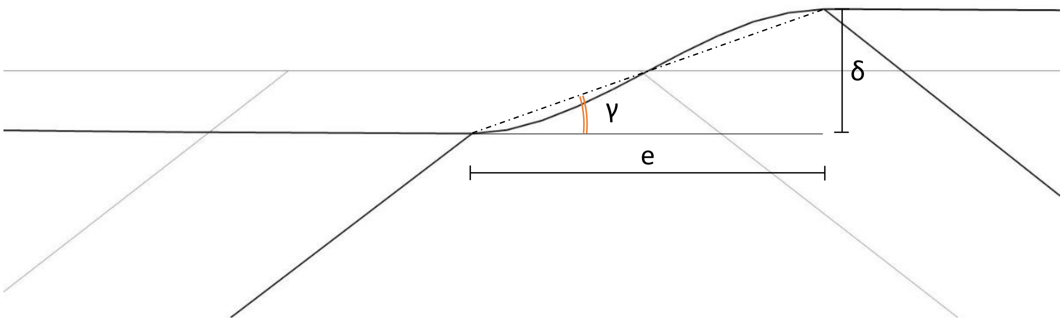


Figure 49: Link deformation

$$\delta \; = e\gamma \qquad (56)$$

It is possible to build a relation between the link and structure deformation in terms of deformation angle as it is shown in equation (57) where L is the total length of the EBF module.

$$\gamma = \frac{L}{e}\vartheta \qquad (57)$$

The maximum base rotation and the maximum tip displacement that the structure undergoes in the deformation process can be evaluated through equation (58) and (59) substituting to γ the maximum rotation allowed by the code [14] for the plastic hinge. As already discussed in Chapter 3, the maximum rotation find in some experimental test is *0.08 rad* and this value is hypothesized to be the collapse rotation value for the structure.

$$\vartheta = \frac{e}{L}\gamma \qquad (58)$$

$$\Delta = \; \gamma\frac{eH}{L} \qquad (59)$$

Knowing the relation between the link rotation and the tip displacement, it is possible to build the curve (Figure 50), as specified in the FEMA code [16], that controls the behavior of the plastic hinge for the simplified module analyzed. Without taking into account the elastic field the main points to be defined, shown in Table 10, are the yielding point, the collapse point and, in addition, three more points are useful for the acceptance criteria.

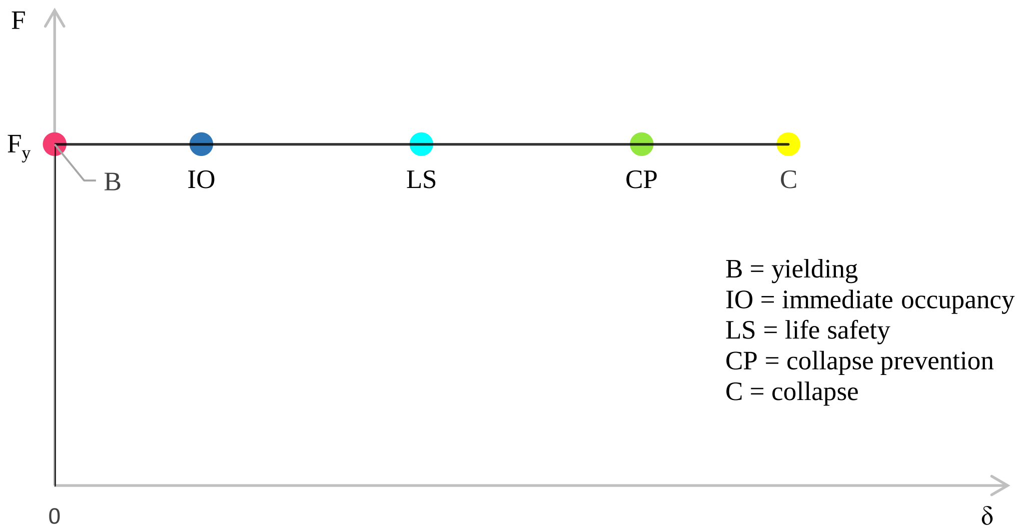


Figure 50: Plastic curve force vs. displacement [16]

The acceptance criteria are defined as a percentage of the collapse rotation, taken as the maximum value allowed by the code.

Table 10: Limit points definitions: acceptance criteria and collapse

CODE (MAX)		
θ	0.016	[rad]
Δ	6.912	in
γ	0.08	[rad]
δ	1.92	in

POINT B		
%ΔL B	0%	
ΔL B	0	in
θ	0.000	[rad]
Δ	0.000	in
γ	0	[rad]
δ	0	in

POINT IO		
%ΔL IO	20%	
ΔL IO	0.384	in
θ	0.003	[rad]
Δ	1.382	in
γ	0.016	[rad]
δ	0.384	in

POINT LS		
%ΔL LS	50%	
ΔL LS	0.96	in
θ	0.008	[rad]
Δ	3.456	in
γ	0.04	[rad]
δ	0.96	in

POINT CP		
%ΔL CP	80%	
ΔL CP	1.536	in
θ	0.013	[rad]
Δ	5.530	in
γ	0.064	[rad]
δ	1.536	in

POINT C		
%ΔL B	100%	
ΔL B	1.92	in
θ	0.016	[rad]
Δ	6.912	in
γ	0.08	[rad]
δ	1.92	in

For example Figure 51 shows the definition in *SAP2000* [17] of the plastic hinge inserted on the main link (*link 24*).

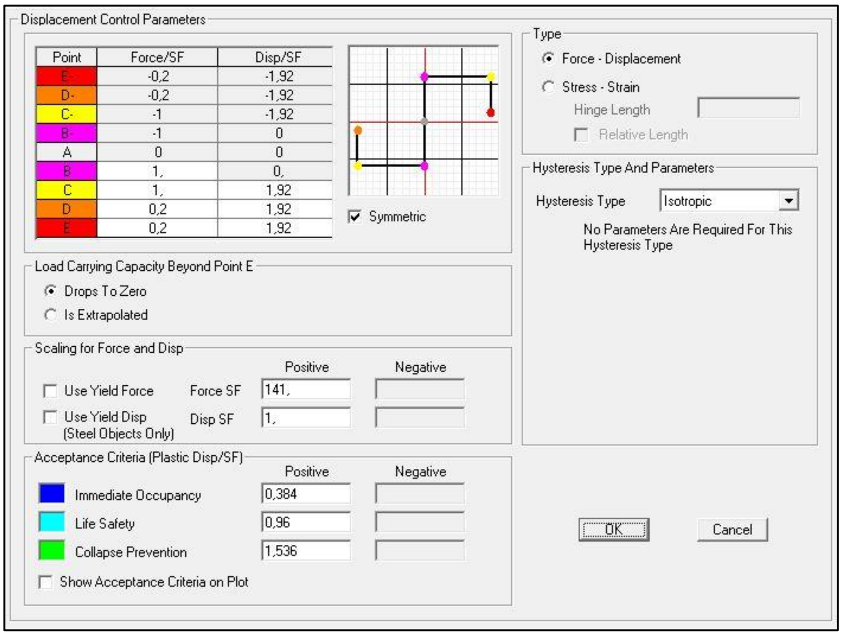


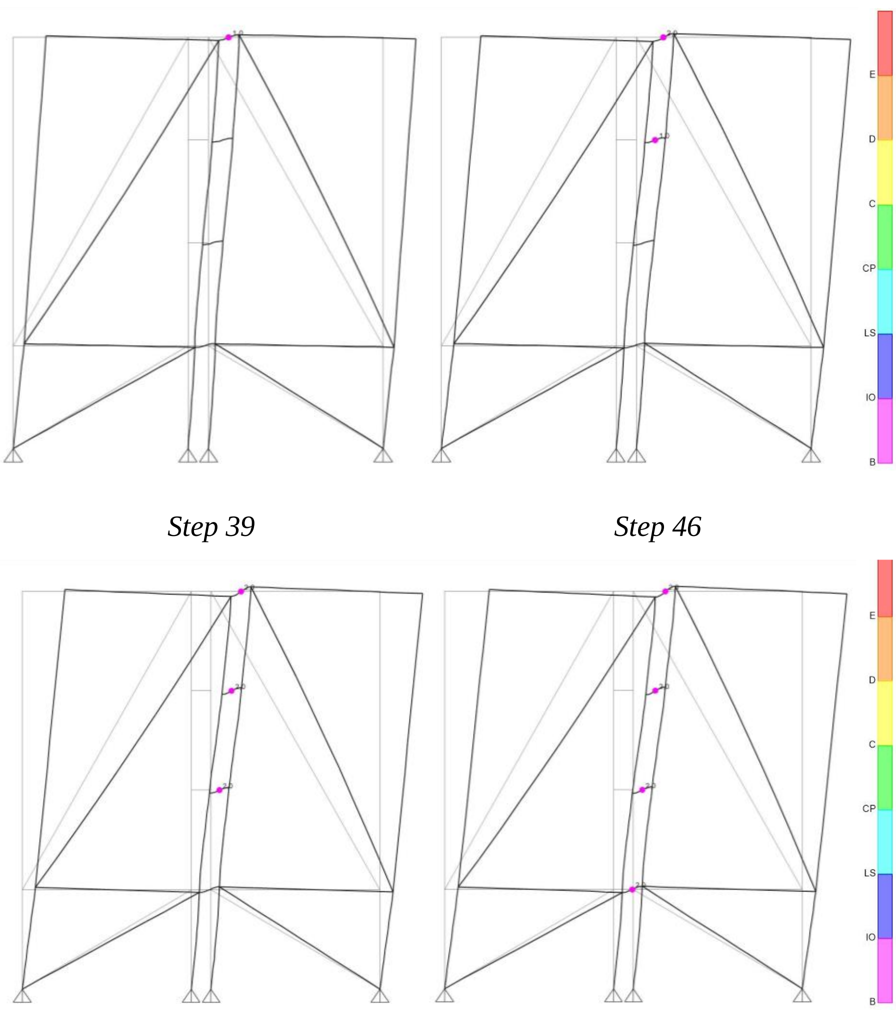
Figure 51: Plastic hinge definition in link 24, dimension in kip-in

4.4.1 Pushover analysis

In order to analyze the lateral load response of the real structure is here considered a procedure that is able to give an accurate and realistic prevision of the lateral loads response of a structure: pushover analysis.

Pushover analysis is a partial and relatively simple intermediate solution to the complex problem of predicting force and deformation demands imposed on structures and their elements by severe ground motion [18]. This methodology consists of "pushing" the structure until it reaches the collapse or the value of a control parameter previously set. The "pushing" is obtained applying in many monotonic incremental steps a defined load path. This procedure is very useful, especially in a design phase, because it describes the behavior of a structure with the simple definition of a mono-dimensional relation between force and displacement. In this way the response of a structure is described through a single degree of freedom system (SDOF) equal to the starting structure. In addition to that, it allows joining the simplicity owning by the static analysis with a reliable evaluation of the structure response in a non-linear field.

In the present analysis, after the definition of the above described plastic hinges, pushover analysis is run in full load control: it means that the structure is pushed through a series of force increment until the lateral load reaches the value of the external load applied. This kind of procedure is different from the standard pushover analysis where the structure is pushed until the collapse is achieved. The software, in a full load control analysis, subdivided the force applied in many multiple load steps and is able to recognize the activation of a plastic hinge creating an additional step for every single activation event. It is always possible to define the minimum and a maximum number of steps or "states" that the software has to create and in this example a minimum of 50 states are imposed to get all the deformation process.



Step 39 *Step 46*

Step 48 *Step 49*

Figure 52: Pushover analysis main steps

Figure 52 shows the main steps in which the plastic hinges are activated: in step 39 there is the activation of the first plastic hinge on the main link. When in the next steps the lateral force increases the shear force will be redistributed on the secondary links that are activated in steps 46 and 48. *Link 3*, the one belonging to the EBF above, is activated as last in step 49. As a result, at the end of all the 51 steps done by the pushover analysis all the plastic hinges are involved by the lateral force. Legend on the right of Figure 52 represents the steps in which the plastic hinge can evolve from the yielding to the collapse according to the parameters set in the previous paragraph.

It is also possible to obtain the trend of the shear force that is plotted in Figure 53 acting on links 24, 17 and 10 from step 20 to 50. When the first link, that is the main link, reaches the plastic shear, the force is redistributed thanks to the column to the secondary links that increase their shear force until they reach their own plastic shear. Once the force reaches the plastic value it cannot increase due to the assumed perfectly elasto-plastic behavior.

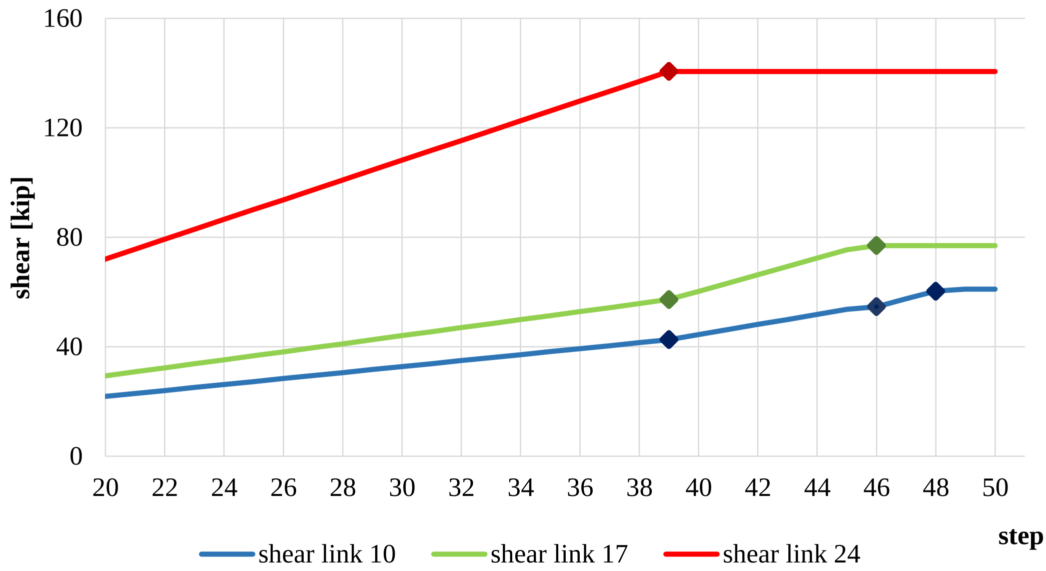


Figure 53: Shear force in links 24, 17 and 10 through steps 20 to 50 (plasticization point highlighted)

From the point of view of the link, the plasticization can be obtained before or after the steps in which now it occurs changing the area of the links, but a very important point to be discussed is the role of the stiffness of the central columns. In fact, a possibility to modify the plastic behavior without changing the links area is to modify the way in which the shear force is transferred to the links.

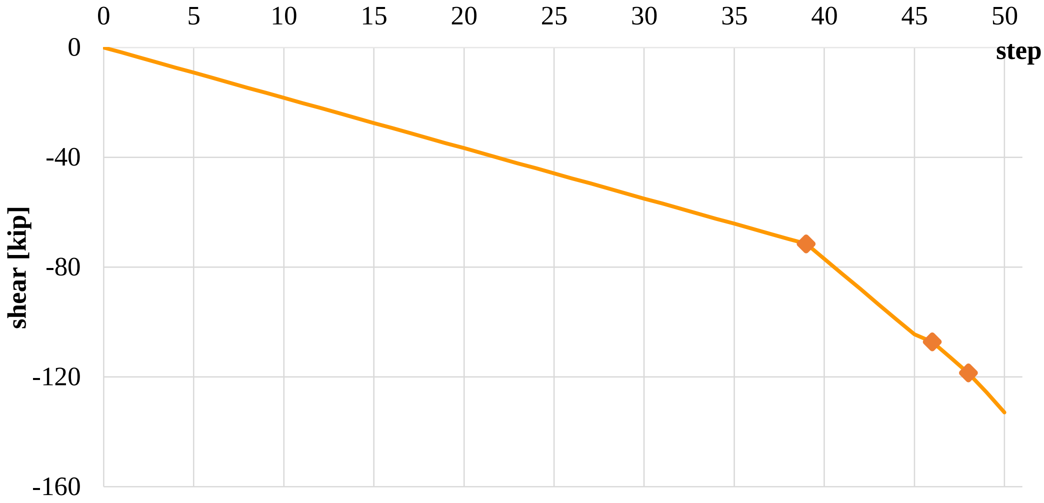


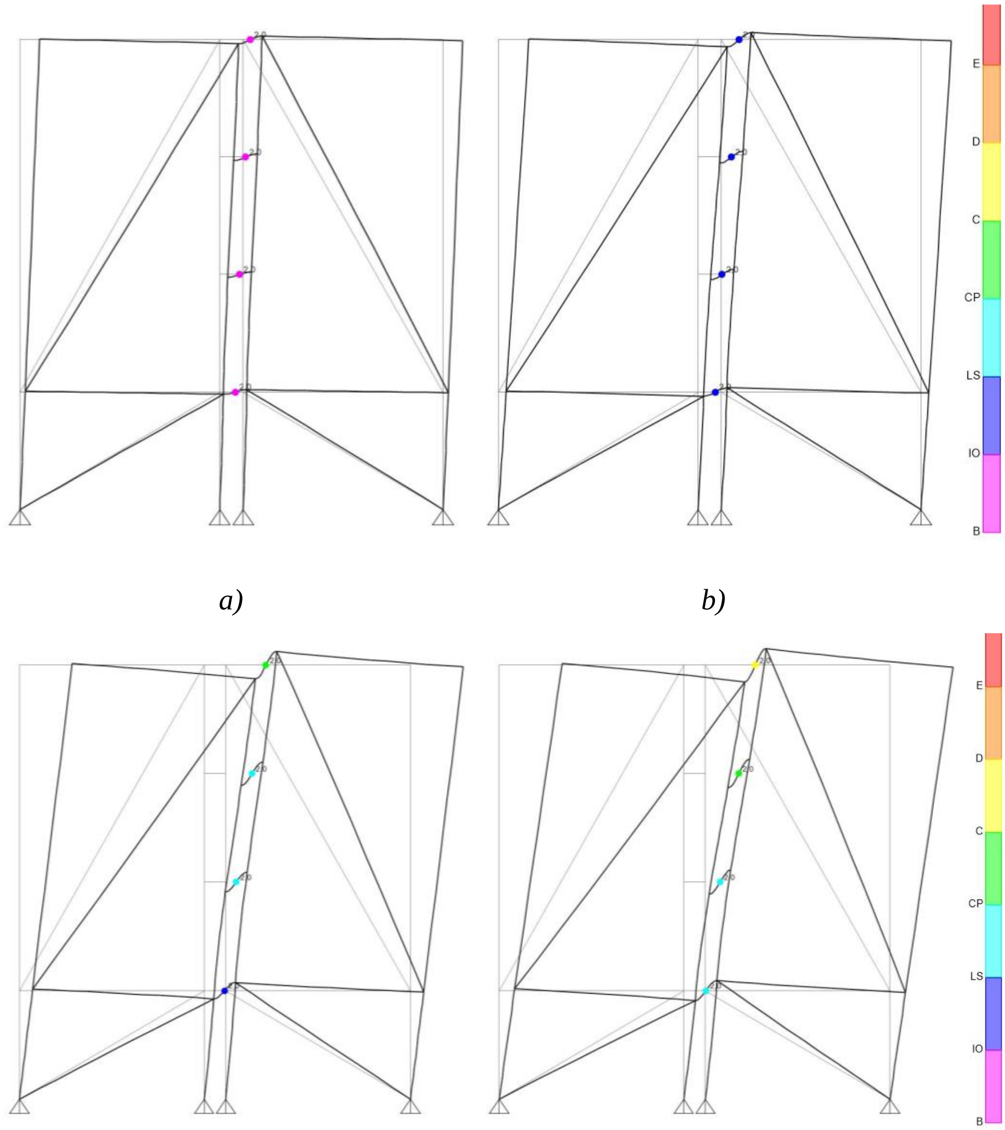
Figure 54: Axial force in central column (plasticization point highlighted)

The trend in Figure 54 highlights the increasing amount of axial force inside the column after the plasticization of the main link. The columns have been sized with the elastic optimization run previously: with their axial and flexural stiffness succeed in involve all the links in the plasticization. Stiffness property is the main point to be discussed both with respect to the links stiffness and the other brace elements stiffness.

4.4.2 Relative stiffness between links and column

With a sensitive study it is shown that if the flexural stiffness of the columns is small with respect to the flexural stiffness of the links, the two columns result too deformable and because of their deformability they are not able to transfer the shear to the links. This effect can by analyzed through a pushover analysis in deformation control where the structure is loaded until the collapse is achieved or a deformation limit is reached. In this case it is sufficient to set the maximum tip displacement (*4 in* in this example) and consider the deformation until one of the main links reaches the collapse.

First of all in Figure 55 is represented, with several steps, the pushover analysis on the structure where the columns are sized with the optimization. It can be noticed that the activation of the plastic hinges of the links results to be almost homogeneous: in Figure 55a) and Figure 55b) all the links participate with the same level of plasticization in the same time. In Figure 55c) the main link deforms more than the others reaching the collapse in Figure 55d) when on the secondary link the level of plasticization is smaller. This is a good behavior because all links are involved with high levels of plasticization and the sequence in which the hinges enter in the plastic field is reasonable.



a) b)

c) d)

Figure 55: Pushover analysis result with columns stiffness coming from optimization

The result, when the columns are too flexible, is shown in the same way in Figure 56. In this case the shear flows into the columns and goes directly to the brace above without the plasticization of the links (Figure 56a) and Figure 56b)). The deformation of the main link continues to grow without involving the secondary link and in Figure 56c) and in Figure 55d) it is shown the plasticization level when the main link reaches the collapse. The activation is not homogeneous and the sequence of plasticization is not predictable as before.

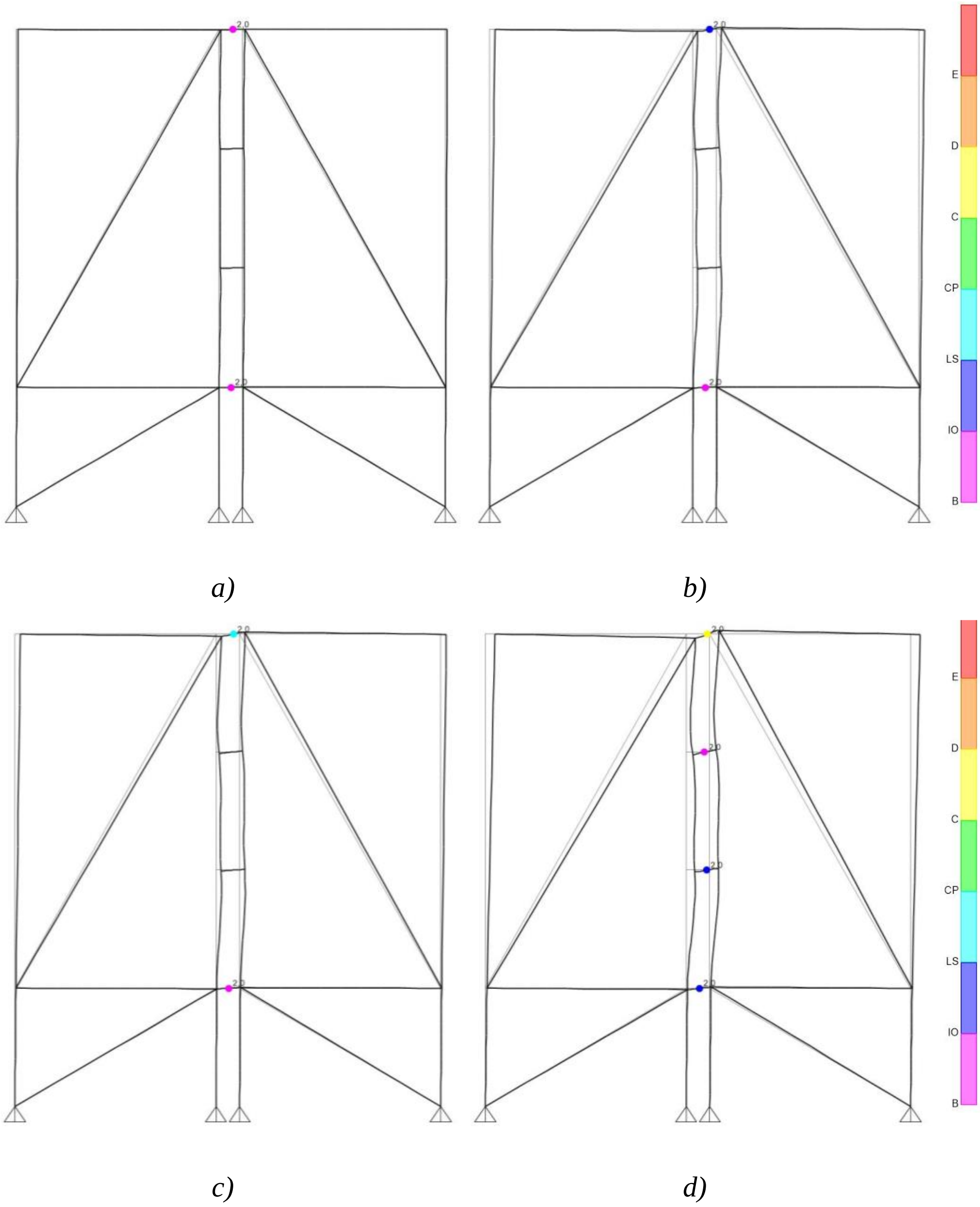


Figure 56: Pushover analysis result with low columns stiffness

In addition, always in Figure 56 it is possible to see the high deformation of the columns due to the low stiffness. In the previous case, the columns have more stiffness than the links and the bending moment associated to the shear force in the links can be sustained by the columns that, therefore, will not have large deformations. These observations can be summed up saying that the columns axial stiffness is necessary to transfer the shear between the links while the flexural stiffness is necessary to react to the bending moment coming from the links without significant columns deformation.

4.4.3 Relative stiffness between diagonal and column

Another important remark has to be done with respect to the relative stiffness between the elements that converge into the node of the main link. As previously said, the shear coming out from the main link has to be taken by the columns that need to have enough axial and flexural stiffness to load the secondary links, but columns too stiff will affect the overall behavior of the structure. From here on the axial and flexural stiffness will not be separated in the analysis assuming that increasing the stiffness means increasing both flexural and axial stiffness.

It can be shown, with another sensitivity study, that if the column stiffness is bigger than the diagonal one (Figure 57), in the global flexural behavior of the structure the diagonals are unloaded and all the bending moment is concentrated inside the two central columns that become the principal elements of the brace resisting frame. Axial, shear and flexural stress states are shown to have a clear understanding of the elements behavior but the most important graph is the bending moment one because it shows that the two columns, being very stiff, can carry all the bending moment unloading the diagonals and the beams. This effect has to be avoided because the brace loses his structural meaning inside the structure: the role of the diagonals is to take the lateral loads through their axial component transferring the force to the brace below, especially to the lateral columns. In other words the flexural behavior of the structure has to be governed by the axial stiffness of the diagonals while the role of the two central columns is mainly to engage the secondary links into the plasticization process.

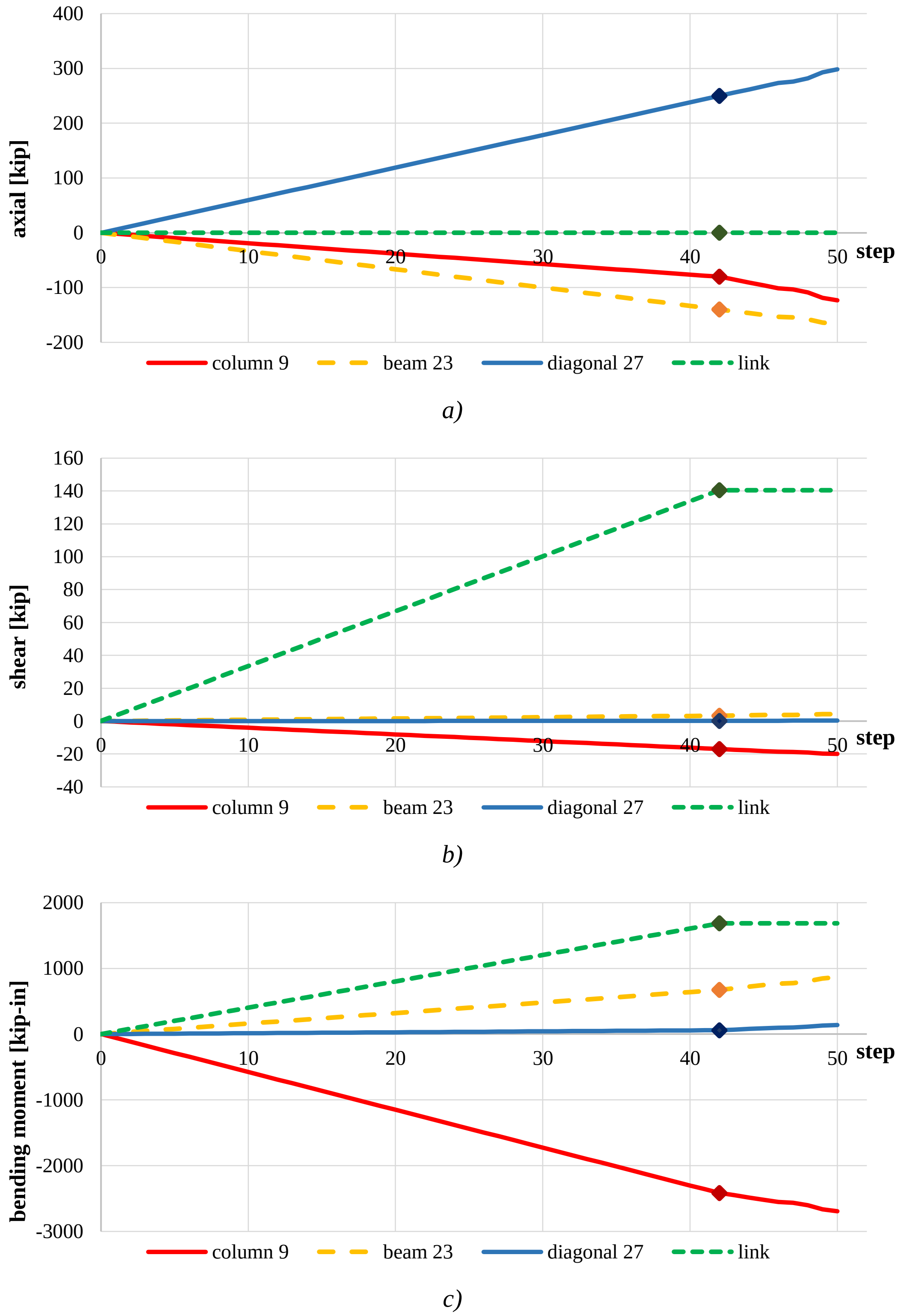
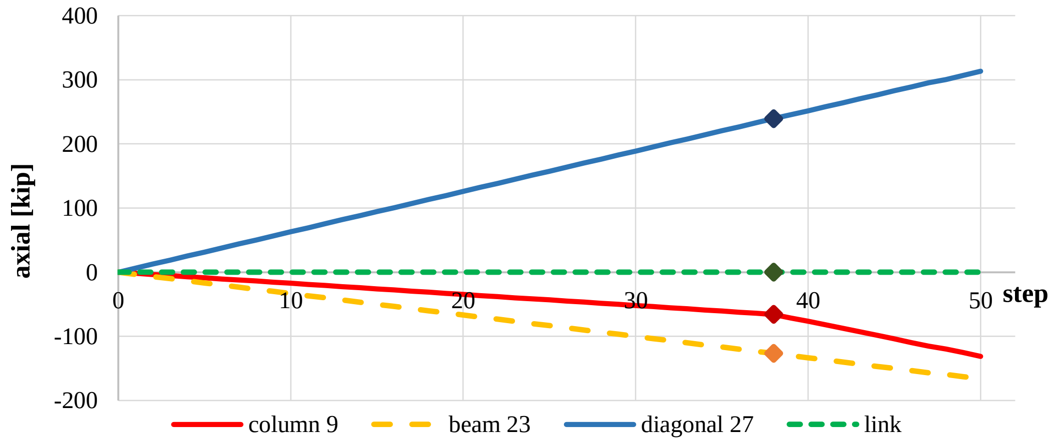


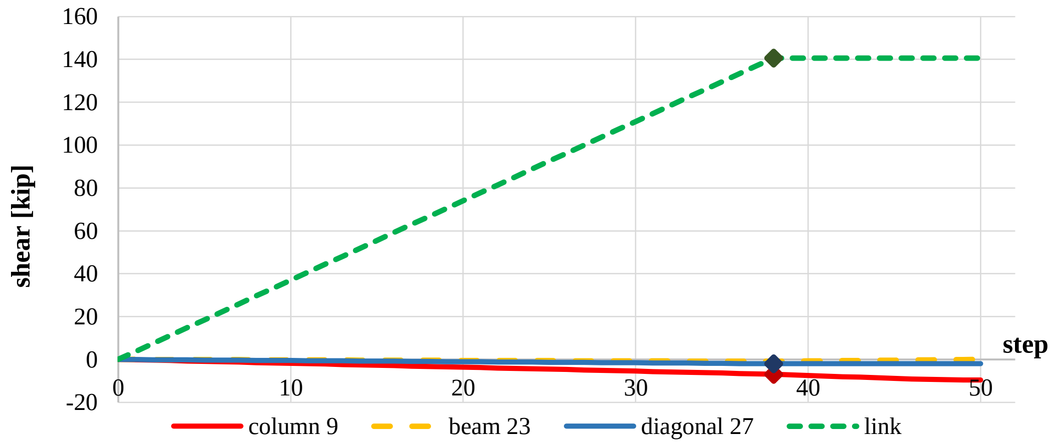
Figure 57: Axial a), shear b), and bending moment c) in the case of column stiffer than diagonals (pushover analysis in full load control)

The highlighted points in the graphs above are the ones in which the main link undergoes the plasticization. They are always shown because, after the plastic hinge is activated, the increasing shear force that cannot be carried by the link modifies the stress state of the brace. The aim is to control it to obtain the best structural behavior

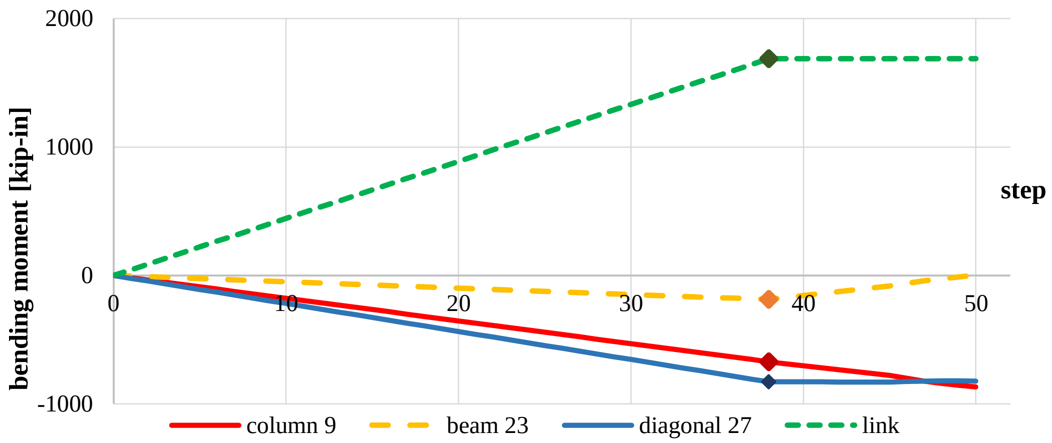
To achieve this goal the stiffness of the columns has to be smaller than the diagonals but at the same time bigger than the stiffness of the link to avoid large deformation and to be able to involve correctly the link in the plasticization process. In the graphs of Figure 58 the column has half of the diagonal stiffness and the behavior is the one searched.



a)



b)



c)

Figure 58: Axial a), shear b), and bending moment c) in the case of diagonal stiffer than column (pushover analysis in full load control)

In the columns, after the activation of the plastic hinge, the axial force increases more than in the other elements due to the shear force coming from the main link and the shear graph shows the correct behavior of the EBF with the link highly stressed. The bending behavior is the more interesting one: from the point in which the main link reaches the plastic shear value; further, the flexural action inside the diagonals is kept quite constant. Thus, diagonals result to be "protected" during the plasticization process of the main link and also the beams have a similar behavior.

By comparing the trends of bending moment in the two situations (Figure 57c) and Figure 58c) the sign of the bending moment is different: in the first case the moment inside the column is balanced by the three bending moment exiting from beam diagonal and link while in the second case the moment at the end of the link is balanced with the moment of the diagonal beam and column. This means that in the last case beams, columns, and diagonals are working jointly to carry the moment inside the main link.

In conclusion, a way to size a structure combining both optimization and plasticization has been found with the separation of two different systems that can interact each other in a very efficient way. The brace elastically optimized takes advantage of the presence of the columns and of the central links, where the plasticization takes place. The sensitive case proposed tries to show how the stiffness relationship between the various components are very important in order to control the overall deformation of the structure being able to govern the force flowing in each member. Another remark is that a good starting point is the sizing of the overall structure achieved with the optimization where some section are constrained to a certain value to allows these elements to be plasticized under the real lateral load.

5

TALL BUILDING CASE STUDY

The final chapter addresses the design of the tied lateral system of a real tall building. On one side the optimization of the braced system is performed with an in-house software (*OPTimizer*) in which the energy based design formula for general elements is implemented. On the other side, the plastic mechanism concentrated in the links is studied through a pushover analysis with the aim to obtain a plastic behavior as homogeneous as possible, under a rare seismic action. As a result, the structure turns out to be optimally sized in its elastic members and achieves a good plastic behavior thanks to the involvement of all the links in which the plastic hinges are defined. In conclusion, the optimized tied lateral system is able to dissipate energy during the most severe seismic action while behaving elastically during all the other smaller events.

5.1 Introduction

The study presented will focus on the structural aspect of an high-rise office tower developed by Skidmore, Owings & Merrill LLP. The tower is part of a $50{,}770\ m^2$ site development located close to a bay in the south of China.

The proposed development will comprise a $312.0\ m$ tall mixed use tower with office space and luxury condo, a $212.0\ m$ tall mixed use tower with hotel, serviced office and a retail podium, providing a total of $242{,}042\ m^2$ gross floor area above grade.

All buildings will share a four-level deep basement that extend over the whole site accommodating retail, car parking and mechanical space.

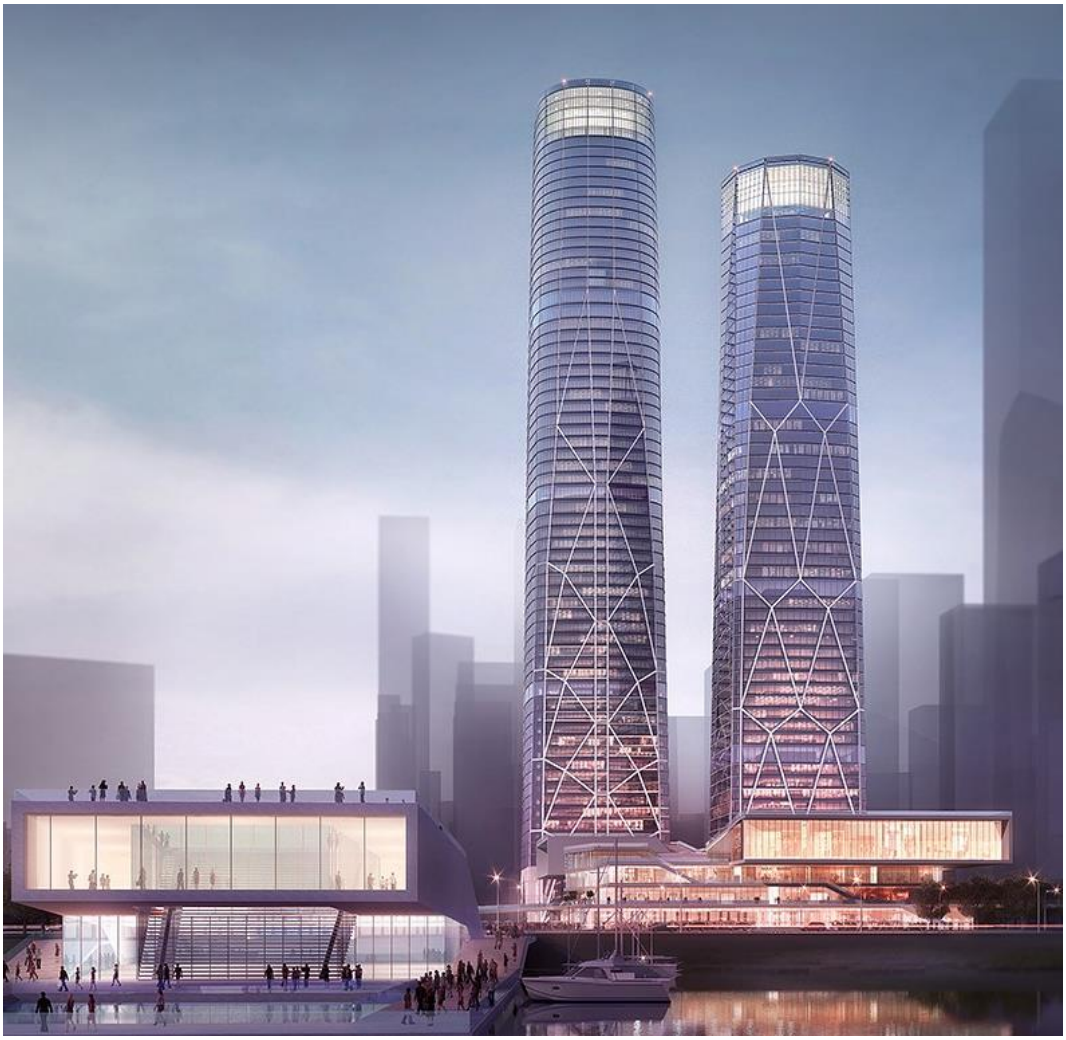


Figure 59: Project rendering. On the left the tower of the case study (image courtesy of Skidmore, Owings & Merrill LLP)

From the beginning of the conceptual design, the goal of architects and engineers was to achieve structural efficiency and architectural value. From a structural view point, the main feature of this project is the particular lateral system configuration that is designed to capitalize on optimization theory and energy dissipation concepts. In this system, the optimal layout is obtained through topology optimization and link fuses are added as dissipative elements. The lateral system obtained by combining the results from the topology optimization with the link fuses is new and innovative.

The resulting geometry is expressed on the façade of the structure and has a significant aesthetic value.

Figure 60: Project rendering (image courtesy of Skidmore, Owings & Merrill LLP)

5.2 Structural concepts

The conceptual design phase of a building can be considered its birth. In this preliminary phase all structural basic ideas are pointed out and, without any detailed calculation, some important concepts and “designing rules” are defined.

In the definition of structural concepts for the competition design of the tower, topology optimization criteria were used by the engineers of Skidmore, Owings & Merrill LLP in order to define the optimal layout of the brace geometry.

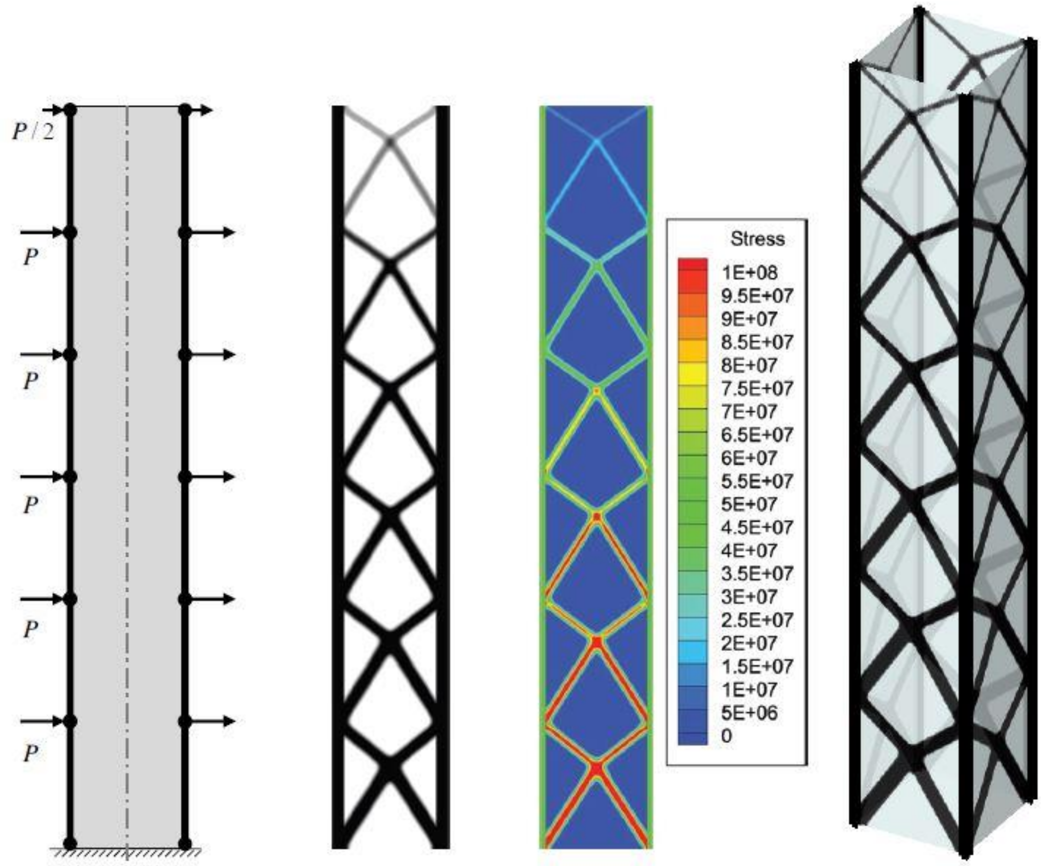


Figure 61: High-rise building topology optimization (image is reproduced from [8])

The problem has been studied by L.L. Stromberg et al. in [8], simplifying an high-rise building with a cantilever with an uniform lateral load distribution along its height. In the example of Figure 61 a two dimensional high-rise building frame, with modules of constant height, is shown. The loading considered is a lateral load applied at each module with a symmetry constraint. As mentioned in Chapter 2, using the combined approach and the SIMP (Solid Isotropic Material with Penalization) theory, the topology optimization problem is run. The braces are completely and clearly defined and the final figure produces the angles expected.

Once this basic layout geometry is deducted, other geometries can be obtained modifying the angle of the brace of the first module and increasing the number of paths (*n*) defining the layout accordingly with mathematic and geometrical aspects as shown in Figure 62.

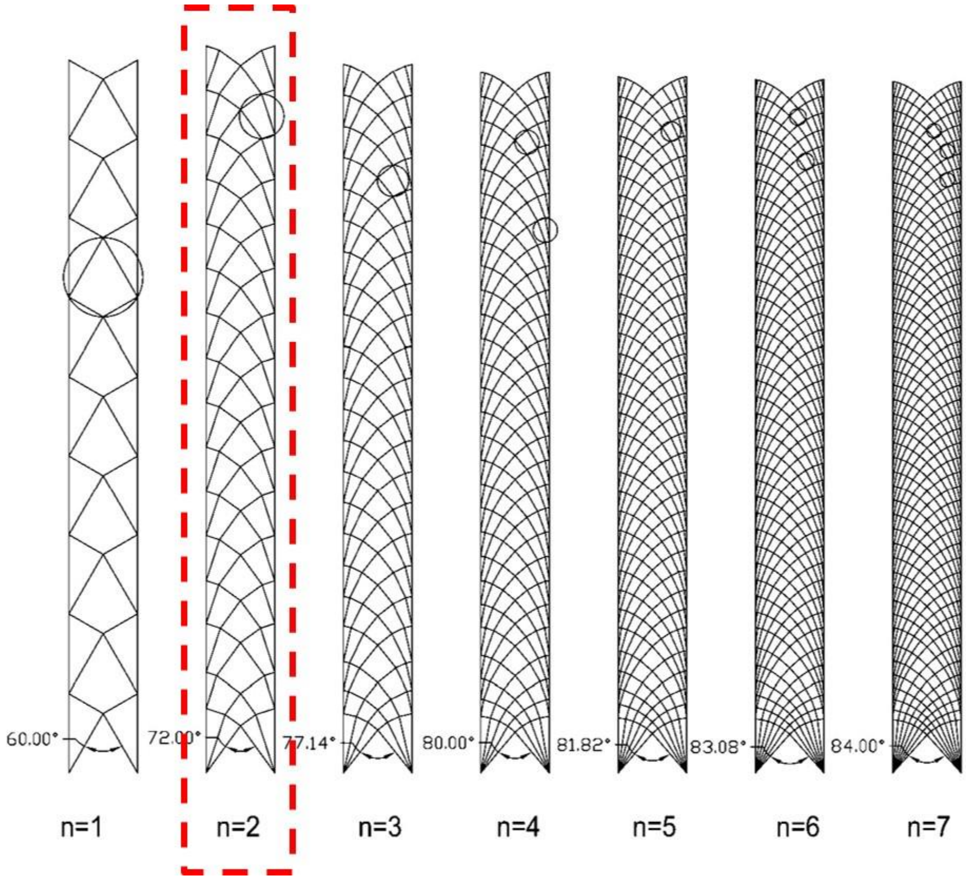


Figure 62: Different optimized layouts (image courtesy of Skidmore, Owings & Merrill LLP)

The layout with *n=2* was selected and chosen as the starting layout configuration. This “starting” layout gives an idea of what would be the optimal layout for a distributed lateral load of constant amplitude. Due to the initial simplification of the loading condition, the final shape could be developed and modified in details to reflect better the real loading condition of a real structure, but these changes would be irrelevant: so it is admissible to keep this "starting" layout throughout all the design phases.

The lateral resisting system will be determinant, associated with the central concrete core, to resist seismic and wind loads forces. However the optimization process conducted in this chapter will focus only in the optimization of the lateral composite resisting frame, without considering the interaction with the core. This is a very important aspect because, giving more importance to the lateral frame, a thinner concrete core can be obtained, saving more gross area space in each floor plan.

This aspect can be studied both in structural and economic terms and some consideration must be done. On equal terms of force intensity acting on these two components of the building, the core and the lateral braced system, the respective percentage of resistance supplied depends on their relative dimensions. It is clear that using a bigger central core, with consequently more material, or having a more efficient lateral resisting frame, the percentage of resistance supplied by each of them changes. If the design focuses on the effectiveness of the lateral resisting system, it is possible to have a thinner core, and this allows engineers and architects to have more usable gross area for each floor, and this aspect increases the value of the entire building. However, it would be cheaper to have a thicker core and a less efficient lateral braced composite frame, due to the ratio of cost between concrete and steel or composite elements. It is an engineering task to find the optimal compromise that leads to the most efficient structural solution and at the same time that produces more free space, in terms of gross area per floor, which reflects in increasing the building value.

On this chapter the attention will be exclusively paid to the lateral braced system, conducing the optimization process constraining the dimensions (thickness) of the central core to the initial dimensions, obtained by a gravity design.

All the previous optimization stages on the braced system were conducted with the hypothesis of elastic behavior of the members, reaching the required stiffness to satisfy the drift target.

In order to give the structure also a plastic behavior, the idea to introduce fuses in the frame layout was introduced. In this way the braced system assumes a dual behavior: elastic and plastic. It is important to point out from the beginning of this study that these two different behavior will be addressed respectively to the brace and to the fuses. In this way the ductile behaviors will be restricted only to the link beams, acting as fuses, preserving the external brace that will continue to behave elastically in all the loading conditions.

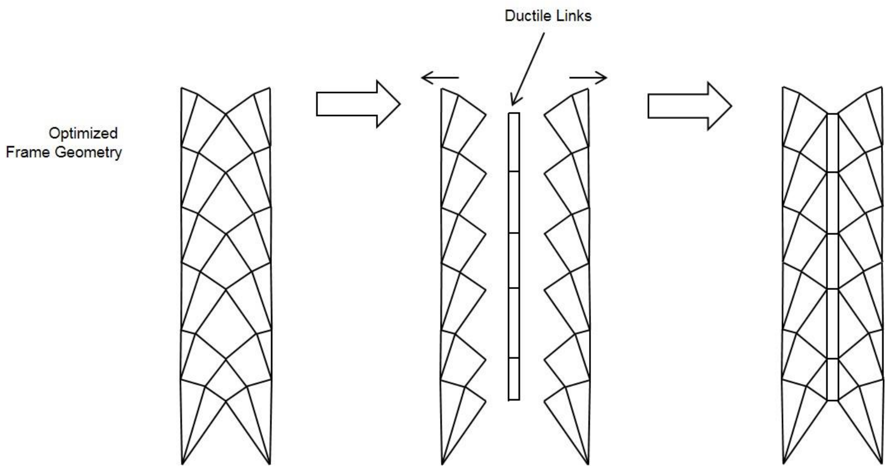


Figure 63: Insertion of ductile links in the braced layout (image courtesy of Skidmore, Owings & Merrill LLP)

In Figure 63 it is clear that the resulting configuration is a new scheme in the lateral braced systems panorama. This new configuration complicates the problem because now the structural behavior changes, ductile behavior has to be considered and also architectural aspects take an important role in terms of proportion and exposition of members. The section shape of each element can be subjected to architectural constraints regarding the impact that the entire frame has in the façade. Connections between link elements and the braced system are now an important issue to be considered in order to allow the structure to perform the expected behavior, while facing with contractor constructability problems.

Once the layout is defined, other two passages were done. First, the tied lateral system was stretched in the vertical direction; second, a moment resisting frame was inserted at the top of the building, where the required resistance is less, as in Figure 64

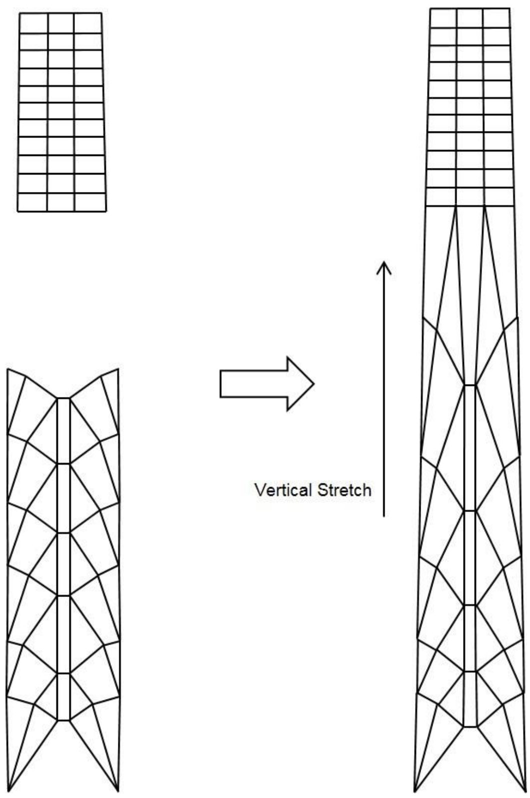


Figure 64: Vertical stretch of the tied lateral system (image courtesy of Skidmore, Owings & Merrill LLP)

Resuming all these previous considerations, the first design conducted by the engineers of SOM for the project competition led to the structural scheme shown in Figure 65 in the next paragraphFigure 65: , where all the structural elements are pointed out.

5.3 Structural system

The 68-story office tower is *299.6 m* above grade to the main roof, and *312.0 m* to the top of the parapet with typical floor to floor height of *4.5 m* (*3.6 m* at the upper levels). The tower has an approximately square footprint, *52.0 m* on the side, and a total above grade gross floor area of *143,207* m^2. The superstructure for the tower consists of a continuous composite concrete/steel braced frame at the building perimeter, and a reinforced concrete core.

Lateral system

The lateral system consists of a continuous composite steel/concrete braced frame at the building perimeter, and reinforced concrete shear walls within the building core. The perimeter braced frame extends from grade to the top of the parapet, transferring to a system of columns and shear walls below grade which will be coordinated with the basement program. The shear walls within the core extend from foundation up to the height of the building, reducing in extent as the size of the core reduces at the upper levels. The shear walls typically range in thickness from *500 mm* to *1300 mm* and utilize C60 concrete. The perimeter frame is composed of concrete filled rectangular tubes (CFT) which will range in size from *700 mm* x *700 mm* to *1700 mm* x *1700 mm*. At the four corners of the tower, pairs of adjacent columns at the adjoining facades will be connected by ductile steel moment frame beams (links) at each floor level. Such links will be sized to remain elastic for wind and frequent seismic loading and to yield at the rare seismic event. A second pair of adjacent columns are placed in the middle of the facades steel brace connected again with links, these columns don’t reach the ground but are connected through diagonals beams to the external base node.

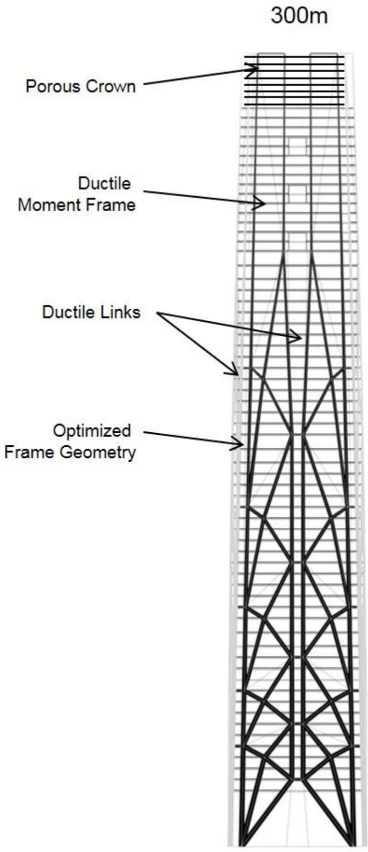


Figure 65: Lateral system elements (image courtesy of Skidmore, Owings & Merrill LLP)

The perimeter braced frame system offers greater structural efficiency since the main lateral load resisting elements are optimally placed at the perimeter, increasing both the lateral and torsional stiffness of the tower. Accordingly, the extent of the shear walls required in the building core is minimized with the resulting benefit of additional floor space, particularly at the upper levels where the core program is diminished and the shear walls transit to reinforced concrete columns and moment resisting frames.

<u>*Gravity system*</u>

The gravity floor framing consists of steel floor framing beams and girders, spanning between the building core and perimeter. The beams support composite metal deck floor slabs, with which they are designed to act compositely. As an alternative, the gravity system could be composed of a reinforced concrete slab spanning between reinforced concrete beams. The perimeter beams in this second option would require a composite steel/reinforced concrete system due to the longer spans. A third option could be a post-tensioned concrete system of beams and slabs. Within the core, the gravity floor framing consists of reinforced concrete beams and slabs.

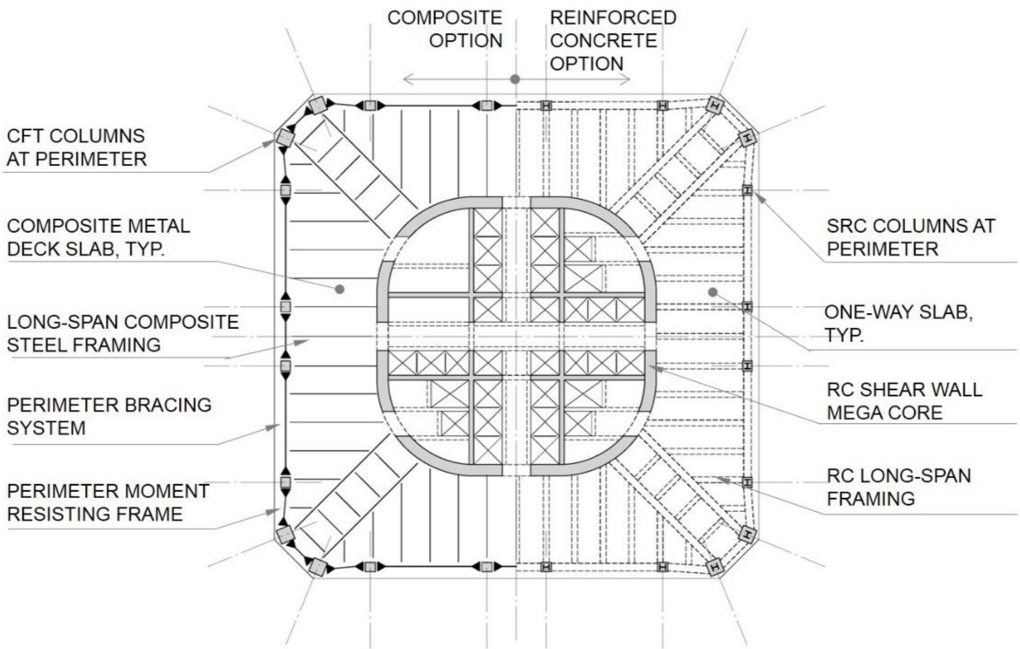


Figure 66: Gravity floor framing (image courtesy of Skidmore, Owings & Merrill LLP)

The continuous perimeter composite braced frame is designed to support gravity loads in addition to lateral loads. Similarly, the reinforced concrete shear walls in the building core support gravity and lateral loads. At the upper levels of the building, where the extent of core walls diminishes, columns are introduced where necessary to support the gravity loads.

<u>*Substructure*</u>

The four level deep below grade substructure consists of conventional reinforced concrete construction, with columns on an *8.4 m* x *8.4 m* grid supporting a system of two-way spanning reinforced concrete beams and slabs. At the office and hotel tower, the perimeter braced frame and core walls above grade transfers to a system of columns and shear walls within the basement levels. Reinforced concrete walls will be provided where necessary to support vehicle access ramps.

Foundations

It is anticipated that the foundation system over the entire site will consist of a reinforced concrete mat foundation of varying thickness, supported on piles extending down to a suitable bearing layer. At locations beyond the towers footprints, the mat will also need to resist hydrostatic uplift pressures, with tension piles or rock anchors provided where necessary. The perimeter foundation wall system comprises conventional reinforced concrete cast-in-place walls. A perimeter waterproofing system will be incorporated around the perimeter of the site.

Design loading criteria and design parameters

In Table 11, Table 12 and Table 13 the main loading criteria are shown (seismic load, wind load and gravity load) and all the value in them are referred to the national code.

Table 11: Seismic load criteria

Design Seismic Intensity	7
Design Seismic Group	1
Basic Seismic Design Acceleration	0.10 g
Site Soil Classification per Geotechnical Report	II
Characteristic Period T_g	0.35 sec
Seismic Fortification Category	B
Maximum Seismic Coefficient α_{max}	0.08

Table 12: Wind load criteria

Basic Wind Pressure: 50 year return	0.75 kN/m^2
Office Tower (50 year return)	0.75 kN/m^2
Podium (50 year return)	0.75 kN/m^2
Site and Ground Roughness Category	A

Table 13: Gravity load criteria

Design Reference period	50 years
Design working life	50 years
Building structure safety level	Level 2
Structure importance factor γ_0	1
Building seismic fortification category	Standard fortification, Category C

Software and assumptions:

The computer analysis model is built using the *Etabs* finite element design package. Shear walls are modeled using “shell elements”. Columns and beams are modeled using “frame elements”. The model is fixed at the base level with pin restraints.

Seismic mass is calculated as a combination of superimposed dead load, 50% of the live load, and self-weight generated from model geometry and material densities. Superimposed dead and live loads are modeled using area loads, line loads or point loads as appropriate. Exterior wall weight is taken into account by assigning line loads at the perimeter of the building.

Wind loads are calculated and input in accordance with Chines codes. Seismic loads are established using the Chinese code based and site-specific spectra, an accidental eccentricity of story mass center (5%) as well as bidirectional seismic actions and CQC method is used for the mode combination.

5.4 OPTimizer software

OPTimizer is an in-house software part of a family of structural optimization tools that have been developed and refined in the last years by Skidmore, Owings & Merrill LLP. The aim of this program is to automate the sizing optimization process, giving the user the possibility to manage with a big structure and a big number of elements in a simply and fast way. This software can be adopted in various moments of the design process and in this application it is used, starting from the structure already illustrated, to find the optimal area of the lateral brace elements.

From a theoretical and a mathematical point of view, the program uses the Energy Based Design Method explained in Chapter 2, interacting directly with the *CSI Etabs* model of a structure. This interaction with a structural software and the properties that are going to be shown give to the user the possibility to avoid some long and time-consuming optimization passages that in the last years were done with spreadsheets calculations. Through its interface, a quite large number of parameters, constraints and simplifications can be set that allow users to have an initial clear understanding of the problem to be optimized.

Depending on the typology of the problem, the program can be utilized in different ways. For example with truss structures, once the geometry and constraints are given, the *OPTimizer* gives as output the final member sizes that satisfy specific targets and specific loading conditions. This is obtained through iterations because of the redundancy of the majority of the structures, but after few iterations the final optimized solution can be achieved. In a more complex problems, such as high-rise buildings, the *OPTimizer* is a very good and useful tool to have an initial understanding of the sizes of each member in the each phase of the design, and it can be used many times accordingly with the development of the project. Indeed, in a structure like those, designers know that geometry, loading conditions and constraints may change during the design phases, and so the initial solution would not be the same as the final one and several iterations and updating would be done during the design.

As said before, in order to perform the sizing optimization process, it is not necessary to export every element of the structure from the structural program by hand and manage this big amount of data manually but, through the API function, (Advanced Programming Interface) of *Etabs* it is possible to get data across to the *OPTimizer* automatically in only one passage. Once data are received by the *OPTimizer*, with some limitations as it will be discussed later, user can achieve, in terms of new sized areas, the desired solution setting in the program loadings, constraints and other parameters. However, in terms of work-time, the user has to update by hand the new optimized model with the new optimized area sections.

The updating of the new optimized model is an important phase of the process in which engineering judgment, about the results obtained with the program, is very important. New sections respond only to energetic design criteria and displacement targets set previously, so it is very important to understand deeply what the new area values mean. If some elements reach a minimal optimum area it means that they are useless to reach the target displacement under the applied load condition and they could be theoretically removed. In practice, remove an element is an operation that has to be done very carefully because the optimization is referred always to a specific load case with the consequently depending path of forces and an element not important inside the optimization, under a certain condition, could be relevant in others.

On the other hand, when an element turns out to have a very big section it means that is an important member in terms of virtual work done to reach the target displacement required; some section constraints have always to be set in order to guarantee the constructability of the structure.

However, from this cases it is clear that the way in which the *OPTimizer* “speaks” to the user is only in terms of section dimensions and is up to the designer understand the importance of a structural element with respect to the whole structure and to the all possible load conditions.

5.4.1 Software assumptions

In order to use the program correctly and understand the meaning of the result some hypothesis need to be known by the user, as well as the mathematical background behind the theory of the energy based design method used in the *OPTimizer* coding. In addition to the assumption of linear elastic material and the use of the small deflection theory the structure need to be statically determined, weightless and the section radius of inertia need to remain constant; some of this hypothesis are briefly explained above.

<u>*Statically determined structure:*</u>

The program assumes that member forces are not dependent on member stiffness, therefore, "avoiding" the stiffness problem, the virtual work done by each member is reversely proportional to its cross-section area:

$$\delta w_i = \frac{P_i p_i l_i}{E_i A_i} \tag{60}$$

In a statically determined structure, once the first optimization step is done, the target displacement can be "adjusted" simply scaling proportionally the areas of each member, due to the reverse proportionality between virtual work, (and accordingly displacement), and area of each single element. In a redundant structure, it is a user task to keep in mind that, once the optimized sections are updated in the new model, the force distribution in each element would change from the initial one, so also the corresponding virtual work done by each element would change: the reverse proportionality mentioned before is no longer valid.

<u>*Constant radius of inertia:*</u>

The program assumes that the radius of inertia of the initial section and the radius of inertia of the optimized one remain the same. This assumption is related to the relation that exists between the area value and the stiffness associated. With this assumption, the program does not care about the variation of radius of inertia that could occur in a section after the optimization process. It is a user task to consider this important aspect in assigning the new sections in the new model, since the stiffness changing would affect the new solution.

Weightless structure:

In all calculations done by the program, it does not take account of the weight of each member of the structure. The *OPTimizer* does the optimization only considering load cases previously defined in the *Etabs* model. Obviously all elements have to be checked, later, for strength considering also the dead load.

5.4.2 *OPTimizer interface*

Dashboard in Figure 67 displays key information of an optimization while it's running.

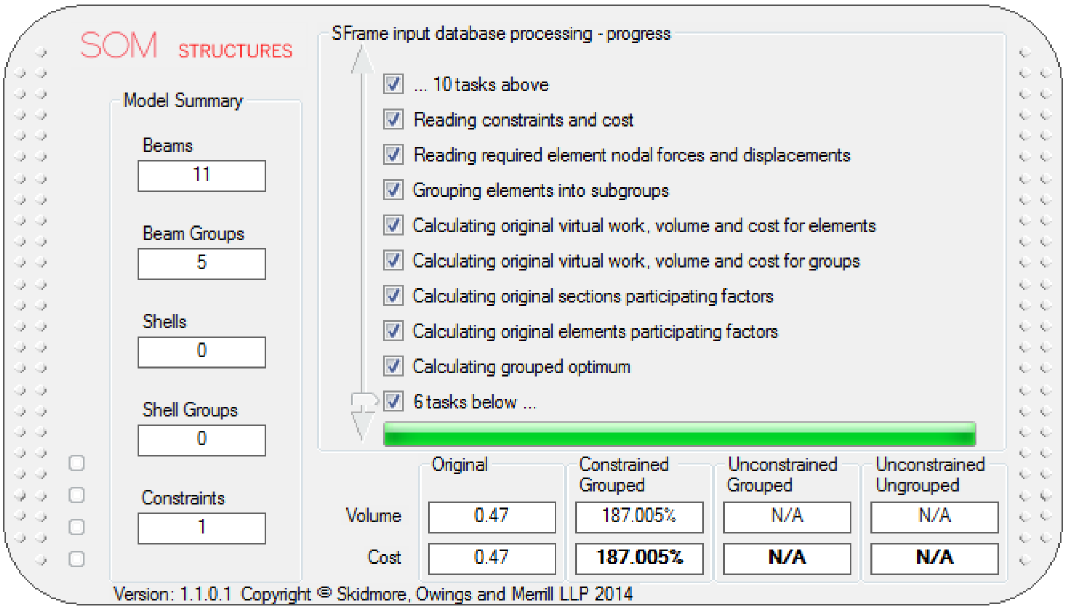


Figure 67: OPTimizer dashboard (image courtesy of Skidmore, Owings & Merrill LLP)

- Model Summary section displays:
- the number of beam/shell elements in a model
- the number of beam/shell groups created
- the number of activated size constraints

- The list lets the user know what step is *OPTimizer* currently working on, what has been completed and what is left to be done.

- Progress bar at the bottom of the list shows the progress of each task.

- Check buttons on the left are used to display the process activity. These buttons indicate how many separate cores are available on the machine. *OPTimizer* will limit the number of separate threads not to exceed number of cores. This is a limitation of available *Etabs* API and *MSAccess* SQL engine.

- Boxes on the bottom display volume and cost values. The meaning of each set of numbers in the group is as follows:
  - Original: unoptimized volume and cost of a structure in a given model.
  - Constrained grouped: optimized volume and cost of an optimized structure where size constrains and element grouping have been applied.
  - Unconstrained grouped: optimized volume and cost of optimized structure where element grouping has been applied but sizes are unconstrained.

Figure 68 displays the main window in which user can set every parameter of the optimization and manage all the elements of the model to be optimized.

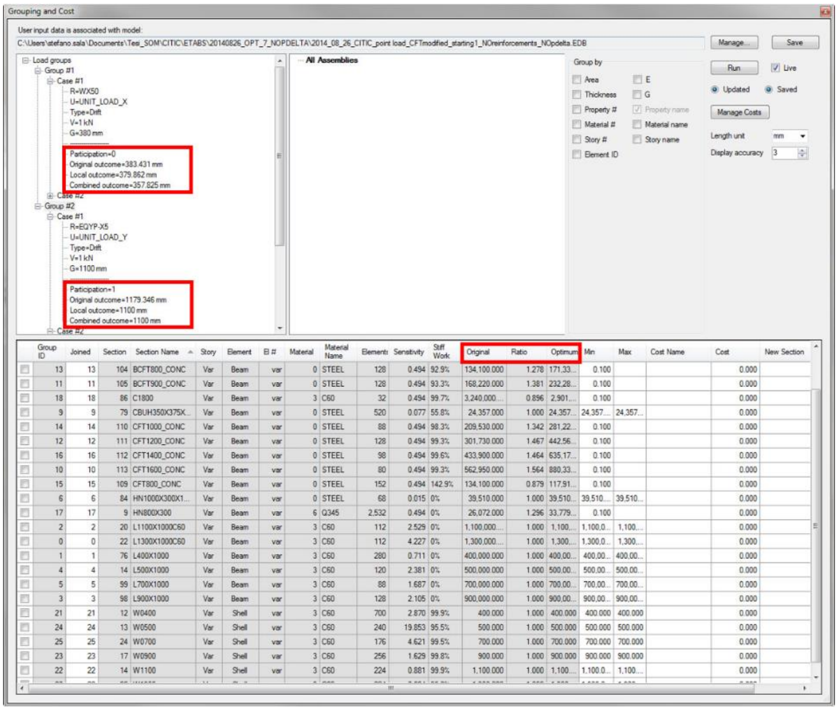


Figure 68: OPTimizer interface (image courtesy of Skidmore, Owings & Merrill LLP)

- Load groups:

- Each Load group in the tree is optimized separately. Each Load case in a group is optimized in multi-objective procedure considering all load cases within the same group.
- Combined outcome in the tree displays the final solution assuming the worst loading situation from each load group.
- Local outcome is an objective value for all load cases within a group optimized in multi-objective optimization.
- Original outcome is an objective value for the given model without changing its original section sizes. This number is a very good representation to check if input data are correct as this number can be easily compared with the one given by the *Etabs* model.

- Columns:

- Group ID: only ID's of all groups are read. Numbers start from 0.
- Section Name: names of sections taken from an input data provided by the user (*Etabs* names).

- Story: story number is available for *Etabs* input data.
- Element: element type (beam/shell). Only one type of elements can be in each group.
- El #: number of the elements if a group contains only one element type, otherwise shows “var”.
- Material: material ID numbers taken from an input data provided by a user (*Etabs*).
- Material name: names of material obtained from an input data provided by a user (*Etabs*).
- Elements: numbers of elements in each group
- Sensitivity: influence of the variation of the element/group area respect to the entire solution.
- Stiff work: % of axial work done by the element/group respect to the total work done by that element/group.
- Original: original section size in selected length units.
- Ratio: optimal to original section size ratio.
- Optimum: optimal size in selected length units.
- Min: minimum size value constraints. For elements with negative virtual work, this value should be greater than zero. Minimum constraint value cannot be less than zero.
- Max: maximum size value constraints.
- Cost Name: names of materials defined inside the *OPTimizer*. Material is defined with unit cost and real (not modeled) stiffness values. Sometimes material stiffness needs to be redefined in the *OPTimizer* especially when composite materials are used. Both unit cost and real stiffness are important for an accurate optimal solution.
- Cost: unit cost of material in units provided by a user.
- Optimum cost: optimum cost of a group. To obtain volume of material for each group this value should be divided by a unit cost.

- Grouping:

Grouping is defined by check boxes, representing grouping categories, inside the *Group by* window. Grouping checkboxes may be grayed out when a grouping category must not be unchecked for proper integrity of the data. It is required that each of the groups contains only one type of elements (beams or shells). None of the groups can contain elements with different original section sizes or material stiffness, therefore a combination of checkboxes will always be grayed out to prevent incorrect selections done by the user.

There is an important difference between selecting *property #* (property number) or *property name* for grouping, which are the most used. As modifying a model, the *property #* may unintentionally be changed by the program, while *property name* is usually assigned by the user and can be more easily controlled.

- Manage data:

Manage data gives access to all saved problems setups. Running model different times with different model name, user can chose setup data from previously run and save model. Running a model under the same name the user data is applied automatically.

Applying previously saved data to a model, user should make sure the two models are equivalent. The *OPTimizer* will try to create load groups tree with load ID numbers from previous model.

In addition, grouping categories will be selected as they were selected for previous model and all element from the new model will be placed into the old groups only if the element properties match properties of elements in these groups from the previous model.

5.4.3 OPTimizer properties

The *OPTimizer* is still under developing, so it is very important to know and be aware of its limitations and possible future applications. Some comments are necessary to be explained in order to use the program correctly and to understand its results.

Handling negative virtual work:

The optimum area section is calculated based on each element/group virtual work. The large virtual work per length value the larger optimum section is going to be. If an element/group has zero virtual work it indicates that the element/group can be removed from the structure without effecting performance of the structure. If an element/group has negative virtual work it indicates that it is working against the required performance and equations result with a cross section area/thickness which is equal to a square root of a negative number, but real sections cannot be negative and also cannot be complex. The smallest section theoretically we could use is zero area.

However, theory used in the *OPTimizer* assumes statically determined structureS. That means that it is assumed member forces are not affected by member sizes. Therefore, selecting zero cross section area for an element carrying any force would result with infinite displacement, which would result in infinite virtual work done by the element, and practical solution would not exist. To prevent this scenario *OPTimizer* assigns minimum limit section value to such element/group and calculates its negative virtual work based on this section. If minimum limit section value is not defined, *OPTimizer* will remove virtual work done by this member/group from the equation and will worn user that part of virtual work has been lost in the process. Losing some of virtual work will produce solutions away from optimal. In order to avoid this problem, user can assign a minimum area value to the element/group that shows this particular "problem". In this way user is modifying the final solution, but this modification is necessary in order to have a good final solution.

Element grouping:

In order to have a more schematic and manageable solution and in order to manage a relative small number of different section name elements in the optimization process, all the element of the structure are grouped in only a few different section name groups, divided with respect to the building elevation. The core elements were divided in six groups, the brace system was divided in four groups and the column system was divided in six groups: each group correspond to a different section name.

This operative choice is reasonable because in that way we can control the number of different sections to be used in the structures, especially in the first phase of the design. Without this choice and importing all the elements in the *OPTimizer*, grouping them for element *number ID*, we would have in the *OPTimizer* all the sections of each of the thousands of different elements of the model to be optimized, obtaining a very big number of different final sections that would have be very difficult to manage. This choice is reasonable also in engineering terms in order to make this phase the design as schematic and simple as possible. Having too many different sections in a single structure is a problem both on the design phase, in terms of time-analysis, and in the increasing of cost for the very big variety of unique-element to be used.

CFT sections:

Since the braced system elements and column elements of the model are CFT sections, it is necessary to convert them in a new single material section in order to be recognized by the *OPTimizer*. Indeed, till now, the *OPTimizer* can "receive" from the *Etabs* model only single plan material sections for the calculation of their stiffness and their virtual work. In order to convert the CFT to a new plane material section we had to face with two problem: area and stiffness of the new sections. In *Etabs*, using the *Frame Section Property Data* form, it is possible to create a new steel section with *General Shape*, with the same section geometric properties of the CFT equivalent section.

Unitary load:

When the unitary loads are set at the top of the structure (at the controlling point), we should distribute it along each of the two principal directions, in order that the sum of each single force applied in each direction gives a unitary load. In the case study there are rigid floor diaphragms, so it is enough if we put a unitary load in a random point of the diaphragm at the controlling elevation. In the case study the unitary load was split in only two concentrated loads: two 0.5 point load in each direction.

Min/max columns:

In the column *min* of the main window one should put a minimum value of area in order to avoid warnings related to negative virtual work computation. Also if the exact solution of the *OPTimizer* would give a null optimum area, one have to put a reasonable value in order to keep a real area section value in the final model. If one assigns null area to a group element, it means that these element/group are deleted by the real model.

In the column *max*, one can set a maximum value for the element/group area. Since we have to take care that the value of optimum area that the *OPTimizer* shows is a reasonable value (it is not too big), one can with this column set a maximum area value. If the area value is too big it means that one has to assign to the model a very big (impossible to manage with from a physical point of view) section in order to get the target solution with this section.

5.5 Optimization process

In this section will be illustrated all the passages done in order to perform the optimization process of the building case study. The conclusions obtained in Chapter 4 will be applied and commented, evaluating their validity.

The main aspect to be considered during this application is that the optimization process deals only with the tied lateral system, without taking into account the concrete core and the rigid diaphragms defined in the *Etabs* model. These two elements interact with the tied lateral system in resisting lateral loads modifying the way in which the forces are distributed in the structure. Surely this interaction is an important topic but is not included in this design that focuses only on the behavior of the external system combining elastic optimization and plasticization of links.

The simplified model studied in Chapter 4 can be now substituted by the real frame configuration shown in Figure 69.

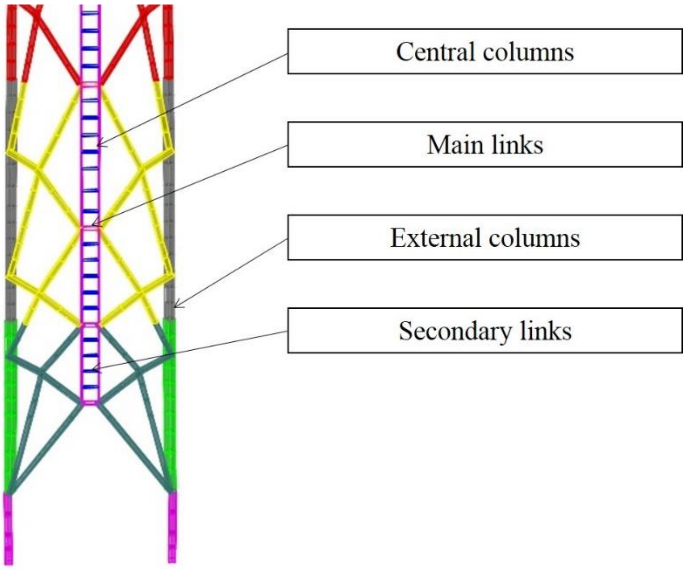


Figure 69: Tied lateral system configuration

In the simplified model of Chapter 4, since the starting structure was created with random sections, all the elements need of an initial sizing optimization to obtain reasonable dimensions to satisfy the target displacement. After that, the attention was focused on central columns and link beams to deal with the plastic behavior.

In the real structure of the case study, a first sections configuration was already obtained by engineers of SOM, dimensioned for gravity and lateral loads. The whole behavior was already satisfactory in general terms (lateral interstory drift and modal periods) but attention was not posed on sizing elements with optimization and no plastic behavior was considered.

It is now possible to subdivide the module elements in two structural categories. On the "stiffness side" external columns and diagonals are sized following the energy based design method: this is reasonable because these elements give the main contribution, together with the concrete core, to the entire building rigidity and govern the lateral resistance, in terms of lateral displacement, of the structure. The optimization process could also be extended to all the elements of the structure but it will be nearly useless.

On the other side, central columns and links (that are already proportioned) are studied outside of the optimization process, concerning with the structure ductile behavior. It has been highlighted that central columns and link beams influence insignificantly the lateral displacement of the structure under lateral loads, therefore their sections are changed regardless to their influence on it. However, at the end of the entire process some iterations and checks should be done to confirm again these assumptions. The sizing through elastic optimization process will be conducted with the *OPTimizer* software, while the plastic analysis will be performed through *Etabs*.

At this point it is important to point out that this optimization procedure has not been considered, till now, into the design phase of this case study conducted by the engineers of Skidmore, Owings & Merrill LLP, however is a first attempt in defining a methodology that, if validated, could be followed in the next step in the real design phase of the project.

5.5.1 Elastic optimization

In order to perform the elastic optimization it is necessary to set correctly the *Etabs* model of the building so that it can be read by the *OPTimizer*. As specified in the previous section, at the moment the *OPTimizer* has some limitations in dealing with

Etabs models. The main problem, regarding this situation, concerns the concrete filled tube (CFT) sections that shape the main frame and lateral columns: they have to be made of a single plane material. Once this modification has been done, it is possible to import the file into the *OPTimizer* and start with the sizing optimization.

In Figure 70 the starting building sections configuration is explained.

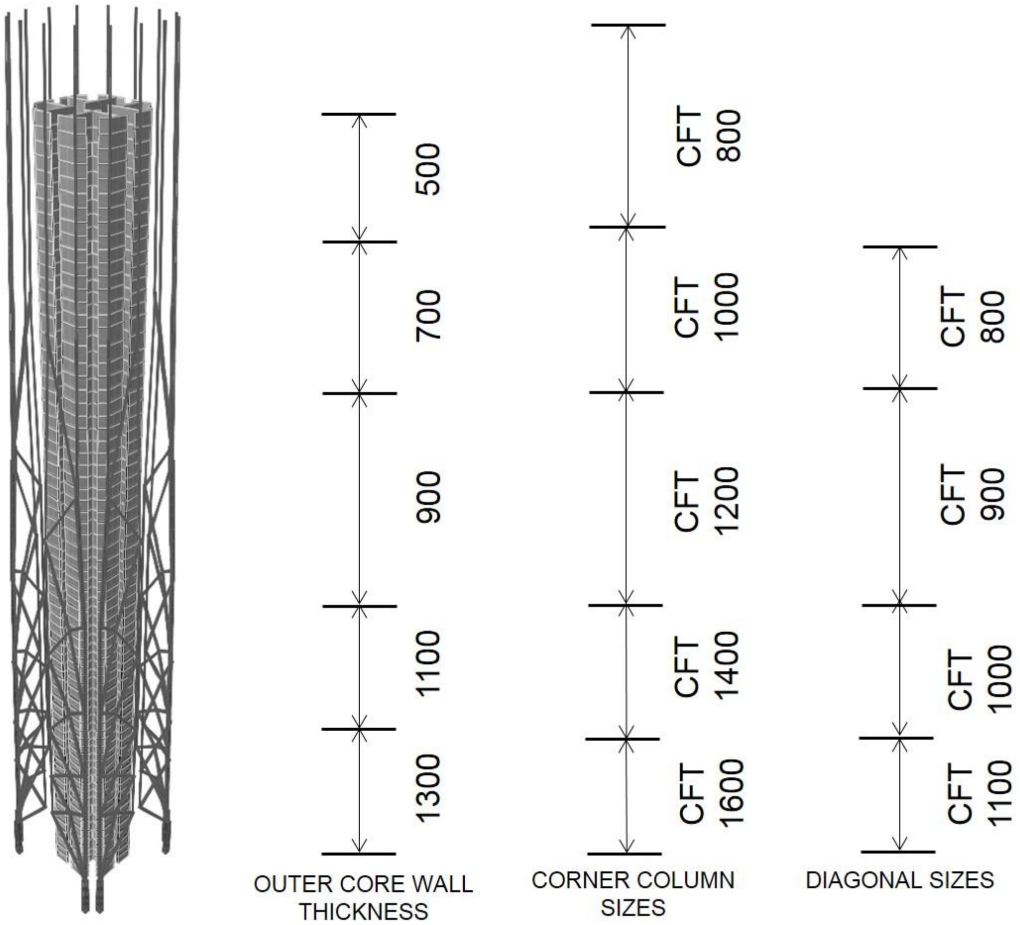


Figure 70: Starting building sections configuration (image courtesy of Skidmore, Owings & Merrill LLP)

It is clearly shown that the building is divided in vertical segments, each corresponding to a different core thickness or element size. Obviously the smaller thickness/section are at the top of the building, while at the basement there are the bigger elements: these dimensions were previously obtained by engineers of SOM for gravity loads, wind and frequent seismic load. The idea of dividing the building in segments with different section names is used to simplify the problem and limit the number of different sections to be designed and constructed. In such a complicated structure, in terms on number of elements, the dimensioning phase would be very long and time consuming without this simplification.

In Figure 71 and Figure 72 the performances in term of interstory drift and principal mode periods are shown. These are the main parameters to be considered in the preliminary design of a tall building and they help designers to understand how the entire structure behaves.

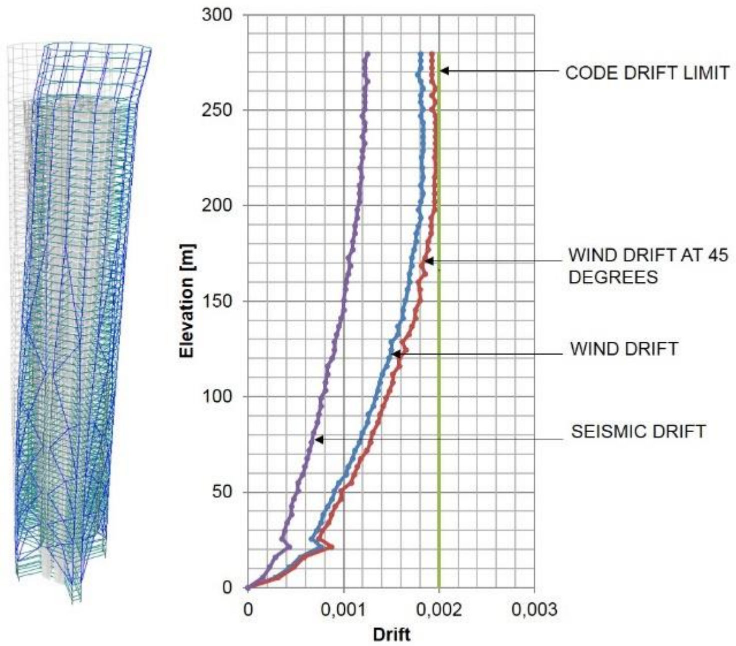


Figure 71: Building interstory drift under different load cases (image courtesy of Skidmore, Owings & Merrill LLP)

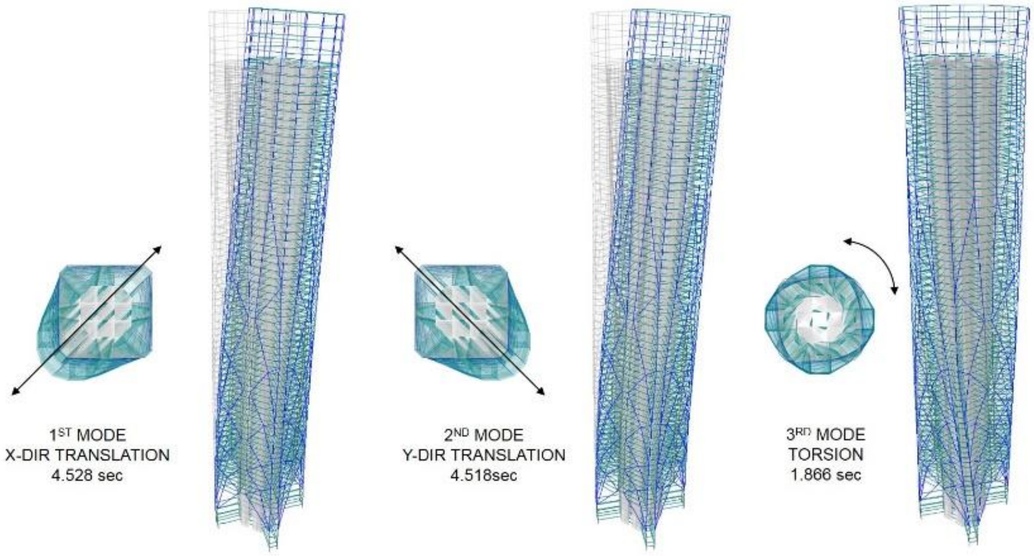


Figure 72: Principal building mode shapes (image courtesy of Skidmore, Owings & Merrill LLP)

In the next steps, it will be considered only the top lateral target displacement as the only optimization parameter. In a more refined analysis also the interstory drift and the principal mode periods should be checked and may become the main controlling parameters during the optimization problem. The period of vibration could be indeed set as well as a target parameter in the *OPTimizer*.

As soon as the *Etabs* model is imported in the *OPTimizer*, some parameters and options have to be set in order to manage correctly the elements to be optimized. First of all the elements, which are initially divided by *element ID number* (obtained from the *Etabs* model), are divided in a different way. The best option, considering the sections division done initially, is to divide all the elements by *section name*. In this way it is possible to deal with only few different subjects and this make the process easier. Another setting regarding the elements is the area constraint. As mentioned before, the choice is to involve in the optimization process only external columns and diagonal elements, therefore an area constraint has to be set for other elements that in this case are beams, central columns and the concrete core. This can be simply done by setting manually as optimal area value the starting area value. In this way only the wanted elements will be optimized.

The last passage concerns about the loading conditions. The loads manageable by the *OPTimizer* are only the load cases previously defined in the *Etabs* model. In this optimization process only the wind load and seismic load are considered and they have different target displacements. For the wind load condition the code limit requires the structure to behave elastically with the following maximum displacement at the top of the building:

$$\Delta_{top,\max(wind)} = \frac{H}{500} \tag{61}$$

On the other hand for seismic frequent load condition the code requires:

$$\Delta_{top,\max(seismic)} = \frac{H}{50} \tag{62}$$

Limitation in equation (62) includes also plastic deformations and ductile behavior. In this case an “approximate” target displacement is used because for the aim of this section it is enough to limit the top displacement of the structure to a realistic value.

As second simplification, an “artificial” seismic load case is used. In order to identify the *rare seismic load*, that should be obtained in this situation following the Chinese Code and all its regulations, the *frequent seismic load* was simply amplified of five times to simulate the rare event. This simplified methodology is obviously not possible in a real design situation, but it is here used only for simplification purpose, knowing that the new load case created is only a qualitative load case, without any code legal soundness.

Once the two load cases, wind and seismic, are set in the program interface, it is possible to decide if optimize the structure in multi-objective optimization or put the two load cases in two different *load group*. In multi-objective optimization, having both load cases in the same *load group*, the program chooses the optimal areas in order to satisfy at the same time the two load cases target displacements. This option is suitable when two or more different load cases are present but belonging to the same typology of action: for example wind load cases in different directions. On the contrary, if different load cases are in different *load group*, the program selects the worst situation between each *load group*, in terms of required optimal area calculated, and assigns these areas to the elements. Therefore the structure is able to satisfy the "worst" target displacement situation, but in respect to the other load cases, all the elements have more area than required: it refers to engineers to decide how to manage with different load cases while running the optimization.

In this study, the wind load case and the seismic rare load case are set in two different *load group*. While the wind load case situation has to be satisfied exclusively in elastic behavior, the target displacement value used for the rare seismic load is composed by two parts: a displacement reached in elastic behavior and a displacement reached due to plastic behavior. Initially the elastic percentage was supposed to be the 0.4% of the admissible displacements.

At this point it is important to highlight that *OPTimizer* deals only with elastic behavior, so it cannot take into consideration the plastic behavior that can occur in the structure: this second behavior has to be studied separately in a different contest. In addition the software doesn't care about strength checks but considers only the target displacement set by the user: in that sense this can be called a "stiffness problem". This aspect can be in a first time accepted since the drift limit is the more stringent parameter to be aware of in designing tall buildings; obviously strength check have to be done in a second passage once the optimization process has already conducted to a solution.

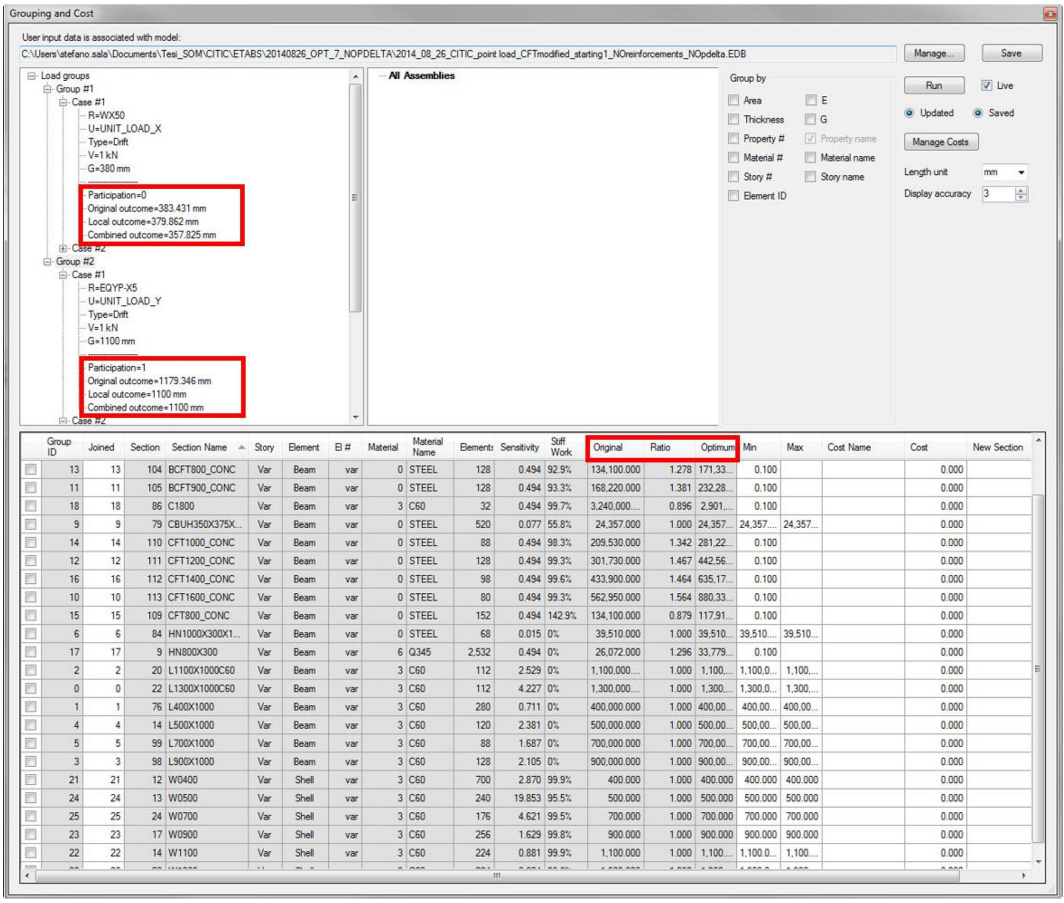


Figure 73 OPTimizer interface (image courtesy of Skidmore, Owings & Merrill LLP)

In the *load group* window in Figure 73, it can be seen that wind and seismic load are in two different *load group*, respectively *group #1* and *group #2*.

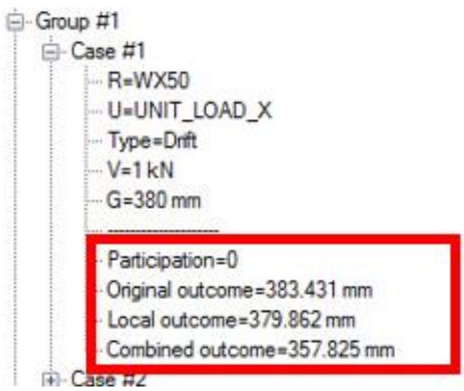


Figure 74 Load group window description

In Figure 74 are presented all the parameters that have to be considered in order understand the *OPTimizer* calculations:

- *R* is the load case name considered in the *case #* of the *group #* taken from the load cases definitions of the *Etabs* model
- *U* is the name of the unitary load case considered. It has to be the unitary load case in the same direction, *X* or *Y*, of the corresponding load case *R*. Their match is fundamental in order to have correct results.
- *Type* is the target parameter used. It can be chosen between drift and modal period.
- *V* is the intensity of the unitary load *U*. For simplicity it is set as *1KN*.
- *G* is the target displacement for the corresponding load case considered in the particular *load case #*.
- *Participation* indicates the importance of the load case. If more *load cases #* are present in a single *load group*, it can assume values in between *0* and *1*, depending on how much important is the *load case* in the multi-objective process. If more *load groups* are present, each of them with only one *load case*, it can assume values *0* or *1*. If 1, it means that the final solution is obtained considering this *load group*, if it is *0* it means that the *load group* is subordinate to the *load group* with 1 as participation value. In the situation of Figure 74 it means that the optimal area values are calculated referring to the *load group #2*, therefore the solution is over conservative if one would consider only the *load group #1*.
- *Original outcome* is the displacement, referred to the load case *R*, obtained from the *Etabs* model. If this number reflects the number shown directly from *Etabs* interface it means that the model has been read correctly. The displacement refers to the point in which the unitary load is applied in the model.
- *Local outcome* is the new displacement that would be obtained if the model is updated with the new areas calculated in the optimization process considering exclusively the particular load case as alone.
- *Combined outcome* has a double meaning. If a multi-objective optimization is run, having multiple *load case #* in a single *load group #,* it will be the displacement exhibited if the model is updated with the new areas calculated considering the multi-objective optimization solution, under the particular

load case U,. If, like in the situation shown in Figure 74, multiple *load group #* are present, it will represent the displacement value, under the particular *load case U,* if the model is updated with the new areas calculated referring to the *load case #* with the *participation value* of *1*.

In the presented situation the *combined outcome* value indicates that the governing situation is the seismic one, in which *local outcome* and *combined outcome* displacement values coincide. As a consequence the structure, under wind load, will exhibit a lower displacement than the admissible one. Therefore it is very important to understand the *OPTimizer* results. In this particular situation it could mean that the target displacement value set for the seismic loading condition, *1100mm,* it is too strict, producing new area values that conduct to a very stiff structure that in wind loading condition exhibits an over conservative behavior.

As can be shown in

Table 15, CFT and BCFT sections are changed, leading to new optimal area values. These sections are respectively the *section name* of external columns and of the diagonals. New sections are bigger than the input ones and this sounds correct because while the input section of Table 14 were dimensioned to satisfy lateral wind and frequent seismic drift limits, now they have to sustain a different lateral loading condition. Indeed, regarding to the seismic load, the target selected is bigger than the wind one, but the intensity of the force which the structure is subjected with seismic load case is higher, so the entire structure is subjected to the strictest loading situation.

Table 14: Input sections

levels	wall thickness (mm)	column	brace
L49 to Top	500	CFT800	-
L38-L48	700	CFT1000	BCFT800
L22-L37	900	CFT1200	BCFT900
L08-L21	1100	CFT1400	BCFT1000
L01-L07	1300	CFT1600	BCFT1100
B1-B4	1300	RC1800	-

Table 15: Output sections

levels	wall thickness (mm)	column	brace
L49 to Top	500	CFT700	-
L38-L48	700	CFT1200	BCFT900
L22-L37	900	CFT1400	BCFT1000
L08-L21	1100	CFT1700	BCFT1100
L01-L07	1300	CFT2000	BCFT1000
B1-B4	1300	RC1700	-

As a remark it can be highlighted that new values of optimal area in Figure 73 have to be "translated" in area of square sections. To do that the area values will change a little bit, modifying the optimal solution given by the *OPTimizer*, however this little change will not affect in a significant way the results. In addition the new sections sound reasonable in terms of dimensions. A limit in the section diameter has indeed to be considered to satisfy physical constructability constraints. If these limit would be exceeded in the optimization process, the user can limit set an area constraint with a maximum area value in the column *max* in the *OPTimizer* interface of Figure 73. In that way the software will limit the area of a *section name* group, "redistributing" the required area in other elements.

Once area results have been commented and accepted, the *Etabs* model has to be updated with the new sections to check if the results of the *OPTimizer* are correct.

Table 16: Top displacements

	Wind top displacement (mm)	Seismic top displacement (mm)
Original Etabs value	383,4	1179,3
OPT target	380	1100
OPT output	357,8	1100
New Etabs value	362	1153

In Table 16 displacements before and after the optimization process are shown. The o*riginal Etabs value* row shows displacement exhibited by the starting structure in the *Etabs* model. It is immediate to see the large difference between the wind and seismic displacements. As target displacements, in the *OPTimizer* are set the values of the second row. The wind target displacement is maintained almost constant while the seismic target one is set around the *0.4%* of the building height. It is now possible to analyze the difference between the *OPTimizer* output displacement value and the displacement obtained in *Etabs* with the new optimized section.

It can be seen that the difference between the two values is very small and it is due essentially to three factors. First of all, the optimal area values and the area values effectively used in the new sections are a little bit different, as explained before. Secondly, in the *Etabs* model rigid diaphragms are present and they modify lightly the entire structure behavior. Finally the seismic load case is applied to the structure with an eccentricity of *5*%, so the seismic loading situation is not completely symmetric and, although the structural system is symmetric, one side of the structure is "more loaded" than the other.

Even though this small differences in displacements, the solution can be considered accurate. Obviously, as mentioned over and over before, engineering judgment is very important in understanding the results of the software.

5.5.2 Plastic design

In this section the central columns and the link beams are analyzed. As specified previously in Chapter 4, link beams can be divided in two categories: main links and secondary links. Main links connect the two parts of the braced system in the intersection points between diagonal members while secondary links are present, one at each story, in between the two central columns. In the previous section has been pointed out that the modification of central columns and link beam elements affect the lateral displacement of the structure in a very small percentage. Therefore in this section it is possible to put aside the "stiffness problem" and focus on a "ductile problem"; nevertheless at the end of the process, in an iterative way, the target displacement has to be verified again.

For the sake of simplicity, in this first part of the design, there are only three different section names to play with: one for the central columns and two for the main and the secondary link beams. At this point it is possible to decide if modify the section properties of the central columns or use the plastic properties of link elements in order to provide the structure of a ductile behavior.

Central columns

Central columns are W-shape sections going all through the height of the building (Table 17). In this design phase they are made by only a unique section but in a second phase the entire columns could be divided to have different sections in different segments of the building height. Considering a W-shape section, gross area value and sectional inertia changes could be studied separately. However it is not always possible because the risk is to create nonrealistic sections with for example problems connected to web-stability.

Table 17: Central column B properties

SECTIONAL PROPERTIES

h	350	mm	F_y	3.44E+02	Mpa
b_w	415	mm	E	2.00E+05	Mpa
t_f	16	mm			
t_w	10	mm			
A	1.65E+04	mm^2			
A_w	3.18E+03	mm^2			
Z_p	2.47E+06	mm^3			
I	1.91E+08	mm^3			

Link beams

Link beams are W-sections too, but their manipulation is subjected to other parameters. First of all, the links present in this building are quite long beams, spanning *4m* for central links and *5m* for lateral links connecting the external columns. Only central link are analyzed in this study and, referring to Chapter 3, the aim is to obtain link beams that yield in shear, thus the equation (46) has to be satisfied. This implies that it is necessary having a section with small plastic shear V_P and a big plastic moment M_P, this is possible, for W-shape sections of same height, mainly with thick flanges and very thin webs. This section configuration leads to two problem. First, the thickness of flanges is limited by constructability parameters, secondly the buckling of the web. Web buckling is a really big problem because it limits the ductile capacity of the section, therefore it is necessary to provide links with stiffeners. AISC [14], in the chapter of EBFs systems, details thickness and spacing of stiffeners in order to guarantee a good ductile behavior. However in this case study the link beam sections created are unique samples, particular details can be provided appositely in order to guarantee a good plastic behavior although the section geometry is very characteristic.

In order to provide a ductile behavior there are two factors to be considered: one related to forces and the other related to displacements. About forces it is necessary that the plastic shear value associated to the section is well established so that each element yields at the right stress level and develops a plastic deformation sufficiently effective. On the other hand it is necessary that the entire structure is able to allow the plastic deformation that occurs in the link beams, because in the case of too stiff structure, no relative displacement in the link beam can be reached.

Since the goal is to involve as more link as possible in the plastic mechanism, it is necessary to understand which is the shear distribution that stresses them. In Figure 75 it is possible to observe that the design shear force in the main links is completely different in magnitude from the design shear force going into secondary links.

This happens because the main links, being connected to the braced system, receive a big amount of forces from the diagonals. The secondary links are instead dependent on the amount of forces that can flow through the central columns. The amount of forces depend on the columns stiffness that influence the nodal equilibrium at the intersection between central columns, diagonals and main links. It is immediate to see that secondary link are subjected to a parabolic trend.

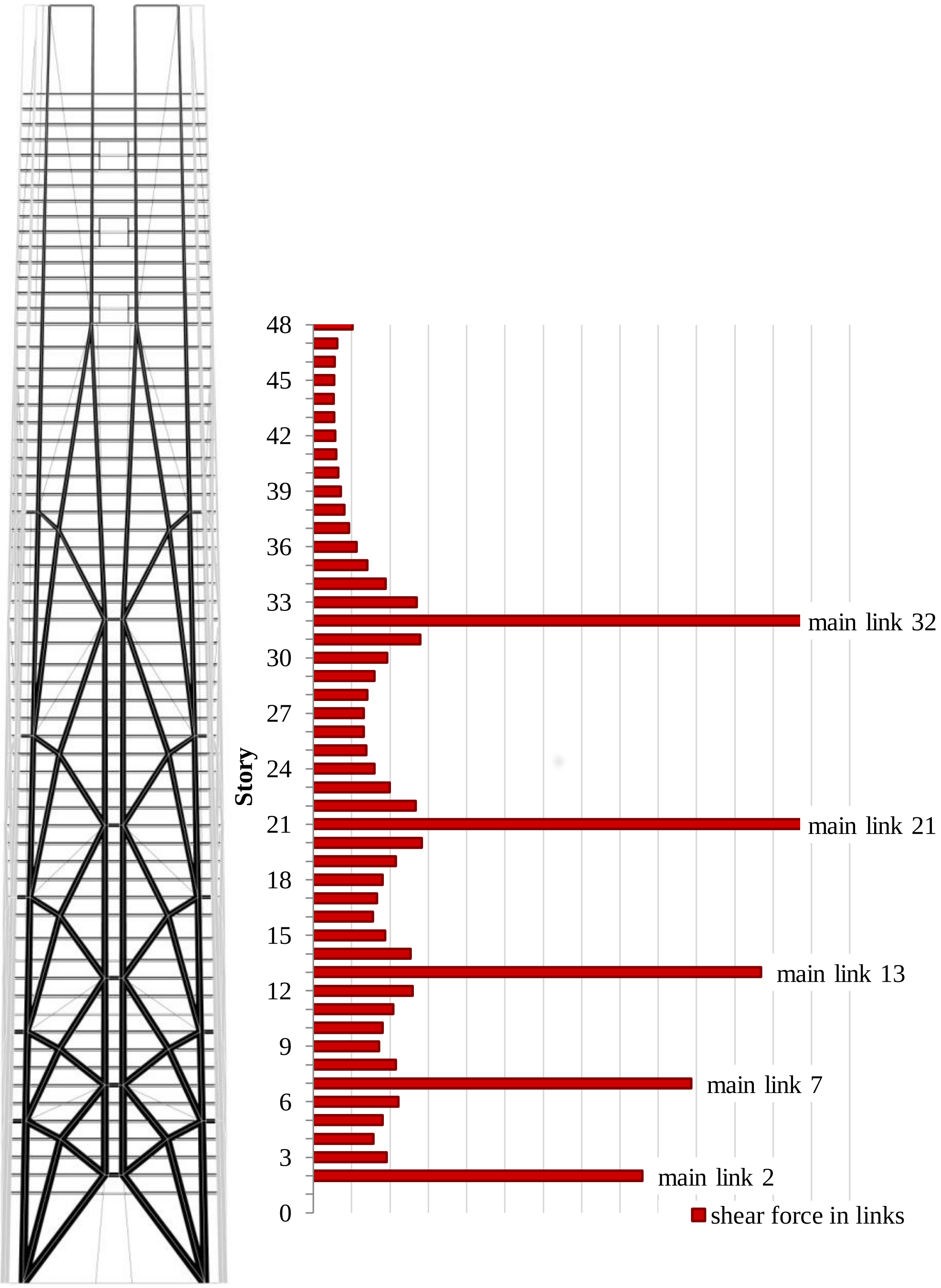


Figure 75: Shear force distribution in main and secondary links

Each secondary link in between the main links is stressed by a different design shear value and this value decreases as the link goes far and far from the main links. As second remark it would be more efficient to have different link sections for each segment of building in between each pair of main links, however for now only one section is considered for secondary links. Having different sections may concerns also the architecture of the tied system since all the links are exposed

The starting section configuration, is shown in Table 18 and in Table 19.

Table 18: Main link A properties

SECTIONAL PROPERTIES

h	1000	mm	F_y	3.45E+02	Mpa
b_w	300	mm	E	2.00E+05	Mpa
t_f	36	mm			
t_w	19	mm	V_p	3.65E+03	KN
			M_p	5.00E+03	KN-m
A	3.92E+04	mm^2			
A_w	1.76E+04	mm^2	$1.6M_p/V_p$	2.19	m
Z_p	1.45E+07	mm^3	$2M_p/V_p$	2.74	m

Table 19: Secondary link A properties

SECTIONAL PROPERTIES

h	800	mm	F_y	3.44E+02	Mpa
b_w	300	mm	E	2.00E+05	Mpa
t_f	26	mm			
t_w	14	mm	V_p	2.16E+03	KN
			M_p	2.75E+03	KN-m
A	2.61E+04	mm^2			
A_w	1.05E+04	mm^2	$1.6M_p/V_p$	2.04	m
Z_p	8.00E+06	mm^3	$2M_p/V_p$	2.55	m

The first modification implemented concerns the change of the link sections, both *main link A* and *secondary link A*. This change is necessary in order to satisfy equation (63) and obtain new sections that are able to yield in shear primarily.

$$e < 1.6\frac{M_p}{V_p} \tag{63}$$

Since links length e are 4m, new sections in Table 20 and in Table 21 are provided satisfying the mentioned relation.

Table 20: Main link B properties

MAIN LINKS PROPERTIES (B)

h	1200	mm	F_y	3.45E+02	Mpa
b_w	400	mm	E	2.00E+05	Mpa
t_f	50	mm			
t_w	16	mm	V_p	3.64E+03	KN
			M_p	9.60E+03	KN-m
A	5.76E+04	mm^2			
A_w	1.76E+04	mm^2	$1.6M_p/V_p$	4.22	m
Z_p	2.78E+07	mm^3	$2M_p/V_p$	5.27	m

Table 21: Secondary link B properties

SECTIONAL PROPERTIES

h	800	mm	F_y	3.44E+02	Mpa
b_w	420	mm	E	2.00E+05	Mpa
t_f	30	mm			
t_w	10	mm	V_p	1.53E+03	KN
			M_p	3.81E+03	KN-m
A	3.26E+04	mm^2			
A_w	7.40E+03	mm^2	$1.6M_p/V_p$	3.99	m
Z_p	1.11E+07	mm^3	$2M_p/V_p$	4.99	m

Once these new two sections are updated in the model, *CASE 1*, plastic hinges are defined in *Etabs* for central link members following the definitions and assumptions used in Chapter 4. Then a pushover analysis, using the rare seismic load case, is performed and in Figure 76 it is possible to see the poor plastic behavior of the structure: plastic deformations are reached only in the main links without any yielding in the secondary links and this is not acceptable since the aim is to engage almost every link, even with small plastic rotation.

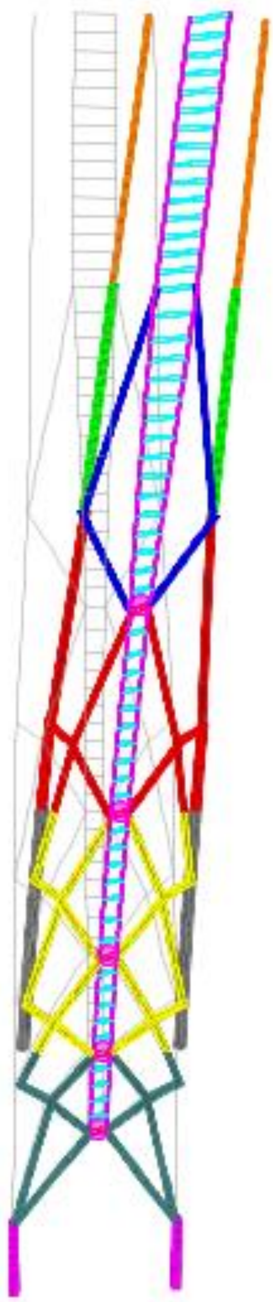


Figure 76 Pushover analysis of CASE 1, plastic hinges activation

In order to involve more links in the plastic behavior, both main link and secondary link sections have to change. Section geometries are modified to have a smaller value of V_P, allowing links to yield with a lower force intensity and consequently to develop bigger plastic rotations. The new sections are shown in Table 22 and Table 23 and they are updated in the model, *CASE 2*.

Table 22: Main link C properties

SECTIONAL PROPERTIES

h	1100	mm	F_y	3.45E+02	Mpa
b_w	400	mm	E	2.00E+05	Mpa
t_f	50	mm			
t_w	14	mm	V_p	2.90E+03	KN
			M_p	8.45E+03	KN-m
A	5.40E+04	mm^2			
A_w	1.40E+04	mm^2	$1.6M_p/V_p$	4.67	m
Z_p	2.45E+07	mm^3	$2M_p/V_p$	5.83	m

Table 23: Secondary links C properties

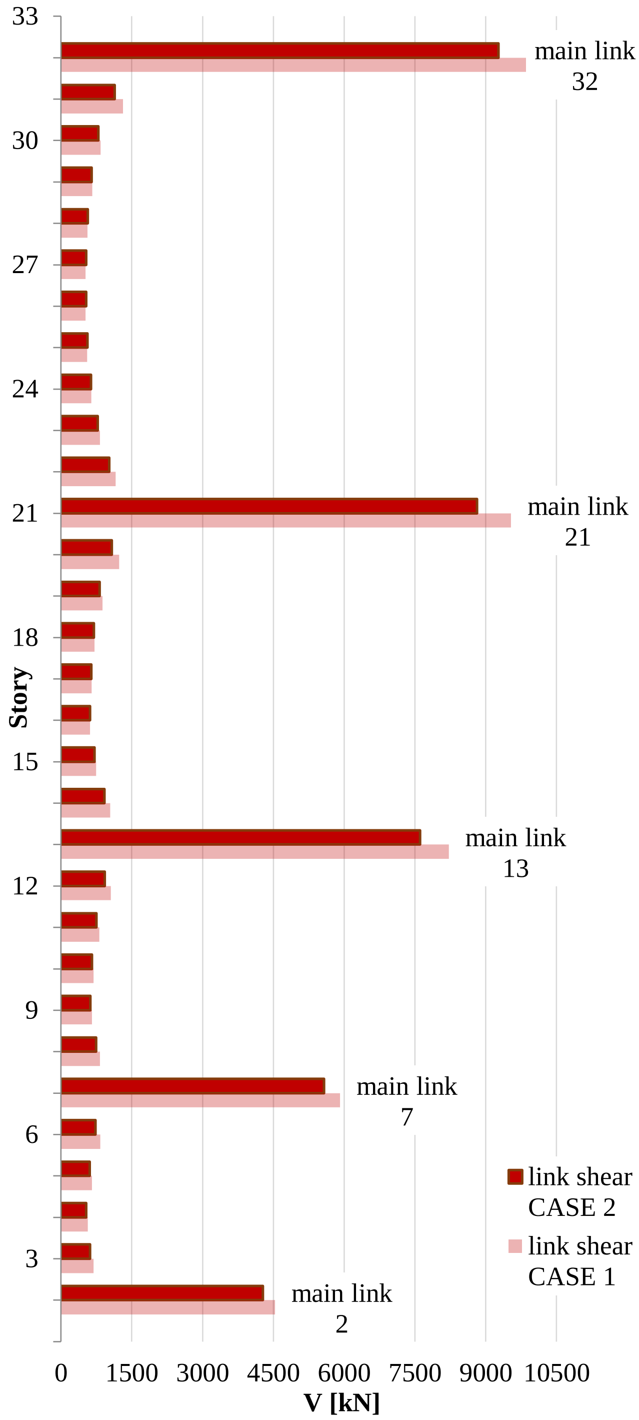
SECTIONAL PROPERTIES

h	580	mm	F_y	3.45E+02	Mpa
b_w	400	mm	E	2.00E+05	Mpa
t_f	36	mm			
t_w	7	mm	V_p	7.36E+02	KN
			M_p	2.86E+03	KN-m
A	3.24E+04	mm^2			
A_w	3.56E+03	mm^2	$1.6M_p/V_p$	6.21	m
Z_p	8.29E+06	mm^3	$2M_p/V_p$	7.77	m

Since these sections changes, the shear elastic distribution through the whole links system is consequently modified, leading to the new values illustrated in Table 24

Table 24: Link shear comparison between CASE 1 and CASE 2

Story number	case 1 |V| [kN]	case 2 |V| [kN]	Δ1-2%
Story32	9975	9270	-7%
Story31	1311	1141	-13%
Story30	841	794	-6%
Story29	660	652	-1%
Story28	560	571	2%
Story27	517	534	3%
Story26	517	532	3%
Story25	555	563	1%
Story24	644	636	-1%
Story23	826	777	-6%
Story22	1158	1020	-12%
Story21	9536	8815	-8%
Story20	1233	1077	-13%
Story19	878	821	-6%
Story18	713	695	-3%
Story17	647	640	-1%
Story16	616	612	-1%
Story15	742	709	-5%
Story14	1042	923	-11%
Story13	8217	7611	-7%
Story12	1058	930	-12%
Story11	811	751	-7%
Story10	691	657	-5%
Story9	656	625	-5%
Story8	829	746	-10%
Story7	5913	5575	-6%
Story6	831	733	-12%
Story5	657	606	-8%
Story4	569	535	-6%
Story3	692	618	-11%
Story2	4535	4280	-6%



In term of forces distribution, the shear intensity in main and secondary links decreases as expected. However in Figure 77 it is clear the difference respect to Figure 76 in terms of plastic hinge formations.

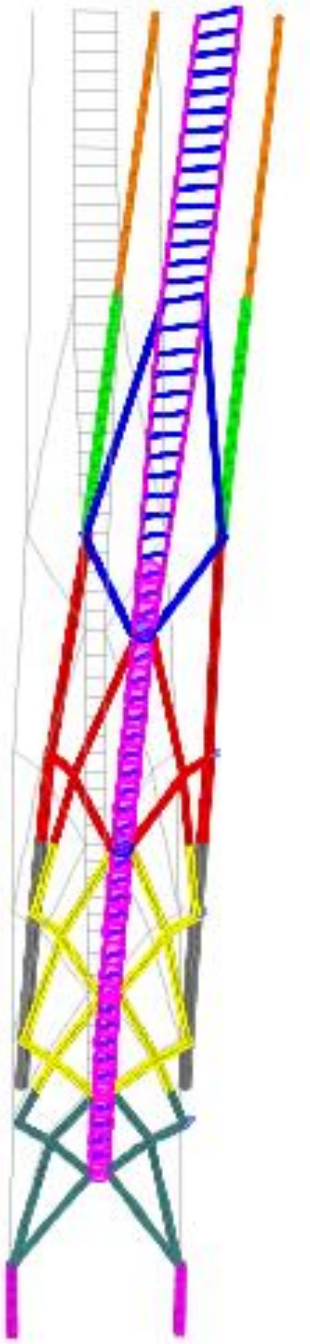


Figure 77 Pushover analysis of CASE 2, plastic hinges activation

Almost every link is involved in the plastic hinges formation, leading to a more uniform and distributed ductile behavior along the height of the building. The most plasticized link is the link 32 which exhibit a plastic rotation of $\gamma = 0{,}018$ but also all the other links, although in a reduced form, actively contribute in dissipating energy.

In this qualitative study the configuration reached can be accepted as satisfactory: every link takes part to the plasticization process, making it uniform. As intended, the main links are more stressed than the secondary ones and they remain the main characters of the dissipation mechanism. A more refined solution could be achieved setting different unique link sections in every link and considering its particular stress level intensity, however this process will excessively increase the cost of the entire system, considering the big amount of particular elements that would have to be constructed and designed exclusively.

Starting from this configuration, a modification of the central column section can be perform to verify its effect on the ductile behavior. The new section in *CASE 3* is shown in Table 25.

Table 25: Central columns C properties

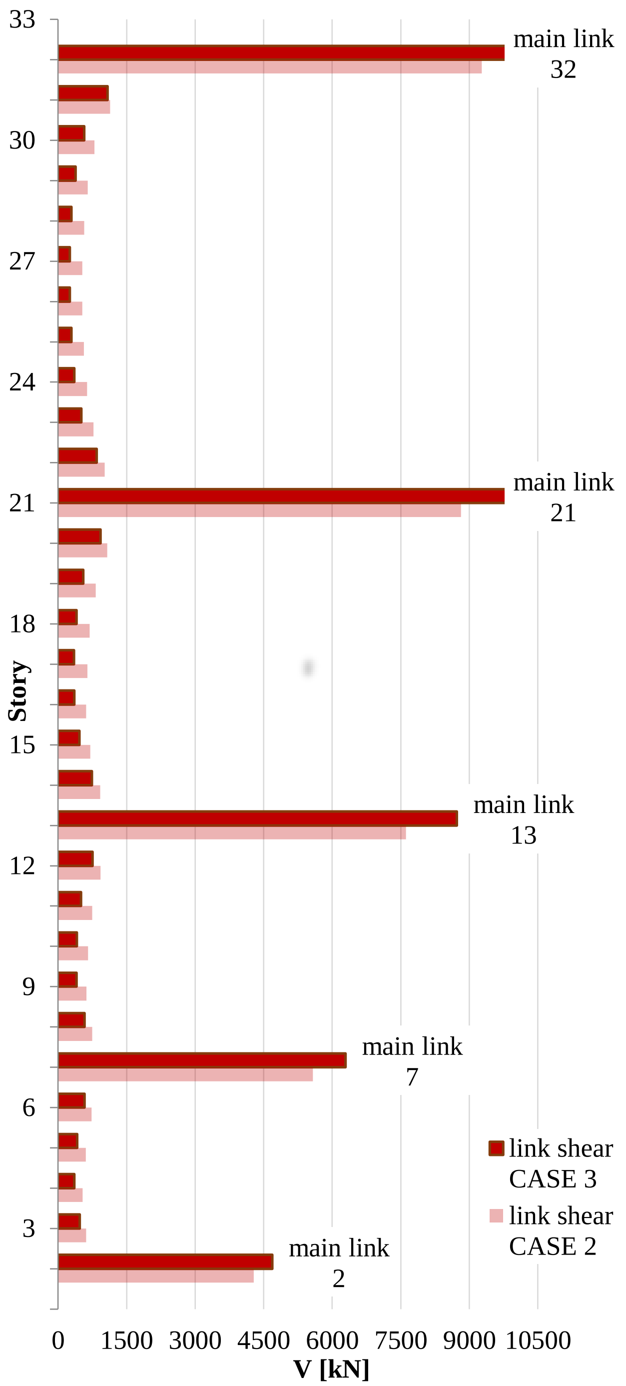
SECTIONAL PROPERTIES

h	350	mm	F_y	3.44E+02	Mpa
b_w	150	mm	E	2.00E+05	Mpa
t_f	12	mm			
t_w	8	mm			
A	6.21E+03	mm^2			
A_w	2.61E+03	mm^2			
Z_p	8.21E+05	mm^3			
I	6.76E+06	mm^3			

Gross area and inertia are reduced in order to see its effect on the link beams. As observed in Chapter 4 the huge decrease of area and inertia have a double effect respectively: stress level intensity in secondary link is extremely reduced as illustrated in Table 26 and the column exhibit now bigger deformation diminishing the secondary links relative displacements, causing no plastic rotation among them.

Table 26: Link shear comparison between CASE 2 and CASE 3

Story number	**case 2** |**V**| [kN]	**case 3** |**V**| [kN]	**$\Delta_{2\text{-}3}$%**
Story32	9270	10990	19%
Story31	1141	1086	-5%
Story30	794	575	-28%
Story29	652	386	-41%
Story28	571	295	-48%
Story27	534	256	-52%
Story26	532	257	-52%
Story25	563	293	-48%
Story24	636	359	-44%
Story23	777	510	-34%
Story22	1020	843	-17%
Story21	8815	10281	17%
Story20	1077	932	-13%
Story19	821	552	-33%
Story18	695	401	-42%
Story17	640	350	-45%
Story16	612	353	-42%
Story15	709	466	-34%
Story14	923	741	-20%
Story13	7611	8727	15%
Story12	930	755	-19%
Story11	751	503	-33%
Story10	657	412	-37%
Story9	625	406	-35%
Story8	746	579	-22%
Story7	5575	6290	13%
Story6	733	582	-21%
Story5	606	416	-31%
Story4	535	356	-33%
Story3	618	474	-23%
Story2	4280	4688	10%



The effect is that secondary links do not form plastic hinges, on the other side the plastic rotation in main links is bigger than *CASE 2* , $\gamma = 0{,}023$, since they have to allow the rotation of central columns. In terms of total structure displacement, *CASE*

3 produces bigger top displacement, but in term of dissipated energy the total behavior is less performant.

CASE 2 is the configuration that better involves all the secondary links. Considerations obtained in Chapter 4 have been validated through *CASE 2* and *CASE 3*, proving that it is important to study carefully the relative stiffness between elements and select a section with a V_P value that can allows plastic hinges formation. At the same time central columns need to have enough gross area and sectional inertia to transfer forces all through secondary links, without overloading main links and diagonals. Results obtained in this section are all referred to a very huge loading condition, obtained with a very simplified methodology that stresses elements with enormous force values. As mentioned at the beginning of this study, this dissertation has a sensitive meaning and more refined calculation, load case definitions and details have to be performed for a more realistic structural analysis.

In addition, in this case study only W-shape beams are used for link beams and central columns. In further analysis could be investigated the utilization of different shape sections and their different behavior.

Concerning the plastic deformation results shown in Chapter 3, obtained with experimental tests, some comments can be done. Values of plastic rotation γ exhibited in these tests were referred to isolated and well stiffened specimens. The situation is different considering an element acting in a real and complex structure: it is highly influenced by the entire structural system in which is included and it is not already detailed, in a preliminary software analysis, in order to capitalize on its plastic displacement capacity.

Concluding this case study chapter, it has to be highlighted that, after this double analysis procedure, iterations are necessaries. Due to the very high redundancy of the building, after the sizing optimization and the modification of central columns and link beams, the force path distribution between elements will slightly change,therefore other iteration would be required to reach a more accurate result. In addition, the structure obtained after the first iteration seems to be very rigid, granting very small displacements for wind load. The consequently lateral displacement is still far below the level of admissibility for wind loading condition, so new target values could be set in order to avoid a too rigid structure. Having a less rigid structure could help also in obtaining more energy dissipation, in case of rare seismic event, due to the greater possibility of the structure to deform.

5.6 Proposal of a workflow for the design

As a conclusion of this case study it is possible to define a series of steps that can be followed in the lateral system design of a tall building joining the results coming both from chapter 4 and 5.

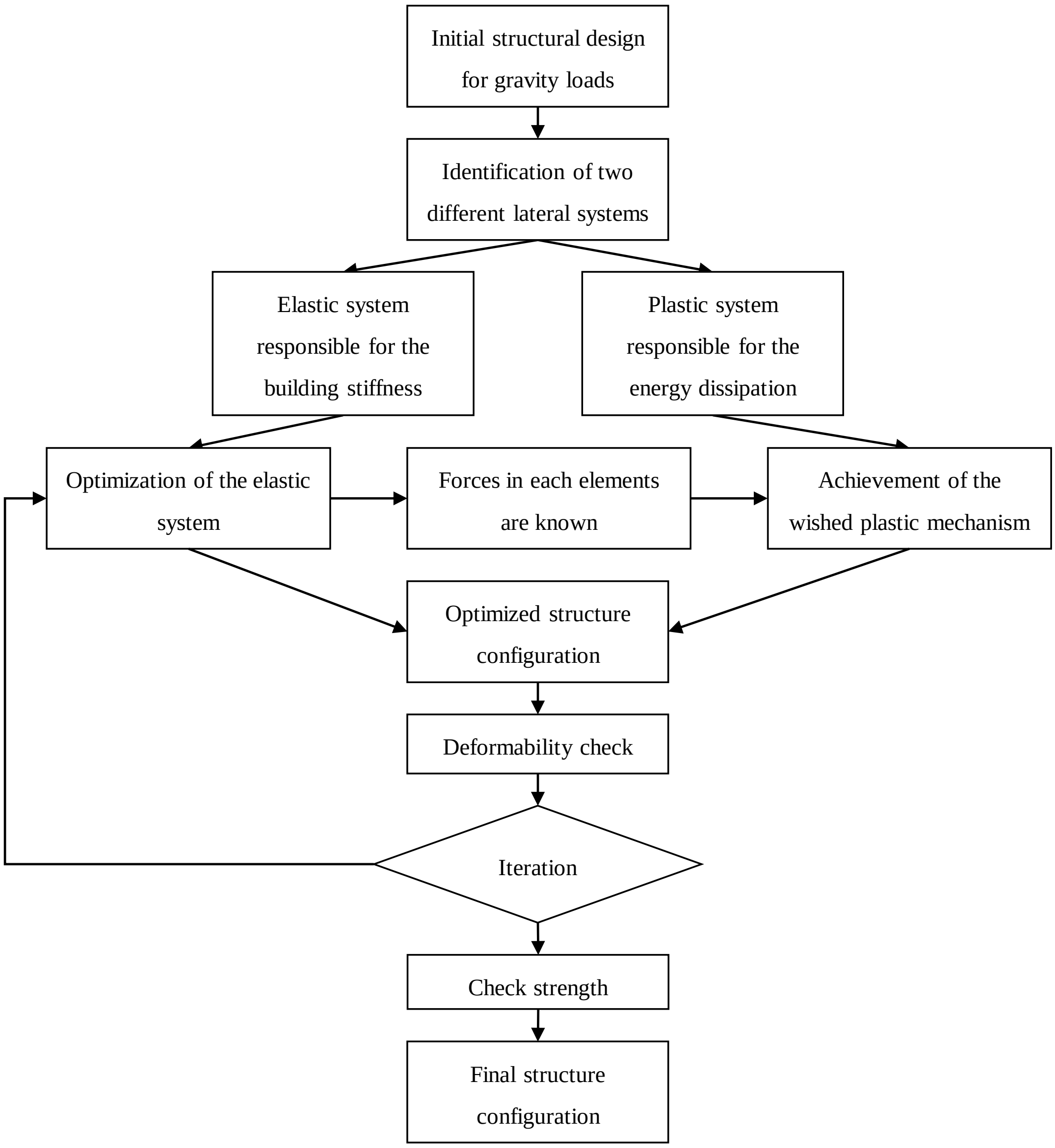


Figure 78: Workflow for the Tied Lateral System design

The workflow, showed in Figure 78, starts when the first schematic design of the structure is available and all the main elements comprising the structure are sized for gravity loads. From this configuration it is possible to understand which are the main resisting elements, such as the cores or the lateral braced frames, and how the different parts of the structure are defined. Knowing, from the structural analysis, the main paths of forces it is possible to distinguish the system that will be optimized remaining elastic from the system in which the plastic behavior will be concentrated. In this identification an important role is played by the way in which the structure has been schematically designed: if the dual structural behavior has been taken into account the identification would be an easy task, conversely if the design doesn't care about this structural concept, the separation between the two systems would be cumbersome. It is easy to verify that the structure of the case study is an example belonging to the first category. In fact the building layout (Figure 63) has been created specifically to allow a dissipation mechanism with the insertion of links and the lateral system geometry comes out from a topology optimization process.

Once the two systems are identified the sizing optimization is run. As widely discussed, the optimized structure will be the one that, with the area distribution found, maximizes the stiffness to reach an imposed displacement. The energy based design method here presented is a valid procedure that can be applied to redundant structure to achieve the size optimization aim. The reached configuration is used to extract the forces acting on the system that will dissipate energy; in the case study, from the optimized structure, it is obtained the shear force distribution acting on links.

As regards the plastic behavior, it has to be defined looking at the geometry of the structure identifying the dissipation mechanism and those elements responsible of such dissipation. This is a very important step because the whole behavior of the structure will be affected by the choice of the dissipation mechanism and then the amount of dissipation required related to the effective behavior of the structure has to be evaluated. Obviously the plastic mechanism has to be carefully selected in order to ensure the building stability during the event that will activate it and at the same time the local verification of the plastic elements has to be checked.

In the evaluation of the dissipation mechanism the theoretical properties might not occur in the same way as they have been studied or tested, but being within a complex structure, they may be subjected to a modified actual behavior. For example, as mentioned before, in the evaluation of the link rotation the experimental tests show a rotation value that in the real structure of the case study cannot be achieve due to the interaction of the lateral resisting frame with the remaining part of the buildings, such as core and rigid diaphragm.

After combining the optimized elements of the lateral resisting frame with the dissipative elements, the overall behavior of the structure needs to be checked again both in the elastic and in the plastic phase. The structure has to be verified under all the loading conditions although optimization ensure the structure to meet a target displacement. In fact the final configuration could have slightly different path of forces because of the sections choice done in the plastic phase. In the case study and in the simple structure of Chapter 4, pushover analysis is performed in order to verify the activation of the plastic mechanism and control the tip displacement of the building. In the case that the deformability check are not verified, the optimization of the elastic part need to be iterated and with the new sizing also the plastic mechanism have to be evaluated again.

In these last steps of the design an iteration procedure can always refine the results obtained from the first optimization especially when some limitation are not verified. The iteration procedure in the case study is not performed mainly because the sections sizing comes from a sensitive study and therefore actions magnitude are huge with respect to any real case, even if the worst seismic condition had been applied. The usefulness of a sensitive study is that allows a quick evaluation of the behavior of the structure, stressing out the main aspect to care about in the whole procedure. Pushover analysis has been used also for this reason, as an effective way to recognize the features of a system involved in a non-linear behavior.

The iterations would consist in repeating the step of the elastic optimization and in the case study this means to use again the *OPTimizer*. If in the first optimization central columns and links were not included at all, while a constrain was imposed on the core section values, now it is possible to have a solution for the whole lateral system with also the area sections of the plastic system constrained inside the process. This method would allow the optimization to consider the presence of a plastic system avoiding the risk of change those section that are sized for the plastic mechanism activation.

As last step the strength level has to be checked in each element. The optimization used in the case study is a stiffness control method and therefore the deformability results to be automatically verified. Usually in tall buildings design the deflection is the governing parameter and the resistance follows as a consequence. Anyway, particular elements such as links or connections could result to be too weak: in this case the section needs to be changed following a strength design and once again these sections can be included in the optimization process as elements with a constrained value.

6

CONCLUSIONS AND EXTENSIONS

The tied lateral system presented in the case study was proven to be an efficient structure that is able to conjugate both elastic and ductile behavior under different loading conditions, leading to an innovative frame layout in tall buildings design.

The workflow studied by means of a sensitivity analysis on a simplified model in Chapter 4 has been validated in a real structure, demonstrating its applicability to the world of tall buildings design.

Diagonals and external columns are the main elements in resisting lateral forces providing the structure with the stiffness required for good behavior both under service life and earthquake-induced lateral loads and they can be studied through sizing optimization in the elastic field. Link beams are instead the elements able to provide energy dissipation in case of rare seismic events and they have to be designed in such a way that the plastic mechanism is as uniform as possible. It is important for this purpose to correctly manage the relative stiffnesses of the different members, with a focus of that of the central columns, which has a fundamental role in engaging link beams.

A first remark can be made with respect to the design of the columns of the lateral system that can be subdivided in two categories: external columns and central columns. Usually the design of a column is based on the gravity load that it has to carry and therefore the main variable to consider is the axial force that flows in it. This is the case of the external columns, which, being connected to the basement, are supposed to transfer their bearing vertical force to the foundation. Conversely, the primary purpose of the central columns is to involve the secondary links in the plasticization process contributing with both axial and flexural stiffness in the transmission of the shear force coming from the main links. Therefore central columns are not mainly designed for gravity load but they are sized in order to fulfill the dissipation mechanism. Beside this fact, also the slope of the diagonals that connect the central columns to the base joints needs to be carefully considered because a change in that value can affect the behavior of the whole lateral system.

The constructability of links and connections between elements is another key factor to be highlighted. Main and secondary links are designed to yield in shear and for this reason their length is strictly related to their cross section shape and to the force acting on them, as discussed in detail in Chapter 3. In the present case study, the design links length e is $4\ m$ and therefore the web needs to be slender to admit high ratio between plastic moment and plastic shear values. This fact leads to have built-up sections in which stiffeners have to be provided to avoid lateral instability and to allow good plastic features such as plastic rotation. The cost related to the production of such links is not negligible in the economy of structural elements and needs to be taken into account in the early stage of the design. Regarding the elements connection the difficulty lies in the fact that the layout is far from being regular, with *90*-degree angles connection. With respect to both diagonals and links, the joints have to be designed with a high level of detailing, which also causes a significant increase in the construction costs.

A general key aspect that should be pointed out is that the optimization hereby proposed is not mainly intended to provide a cost saving process, but a way to obtain an efficient and effective structural behavior. Undoubtedly, structural optimization is generally aimed at saving material, by finding out the best configuration to satisfy a set of pre-imposed requirements, but when it is applied to structural systems with a high degree of complexity, saving material becomes a secondary objective, subordinate to the creation of a highly effective system, with an aesthetical quality. Indeed, the structure to which the optimization is applied it is not an ordinary structure, so that the inherent additional costs due to the creation of a masterpiece with unique architectural features are justified. Therefore the use of optimization in this type of buildings is not carried out with the naïve idea that the structure at the end will result "economic", but that the material will be exploited in all its features and characteristics, both structural and aesthetical.

Pursuing such efficiency, needs to be an effort and a responsibility shared between architecture and engineering, aimed at achieving new modern design concepts coherent with the basic laws of structures.

Some considerations are pointed out as possible studies and further developments, some concerning the elastic optimization phase and the last about a new methodology to be developed in design.

A first study is suggested about the *OPTimizer* software, concerning the possibility to deal with different materials through the sizing optimization process. In the *OPTimizer* interface it is possible to set different materials, and consequently their properties and cost per unit volume, allowing the optimization process to consider the optimal area distribution through different materials in relation to their efficiency, defined as the ratio between their elastic modulus and cost. In this way, it is possible to exploit the more economic materials, where the word economic assumes an important meaning, including both structural efficiency and cost control. Steel beams and concrete elements, as the core, could be included simultaneously in the sizing process and so the "cost manager function" can help indicating the optimal distribution of material, both steel and concrete, in different elements of the same structure. Engineering judgment assumes a fundamental role in defining the scope of application in which the "cost manager function" could have an effective utility. If applied indistinctly to the whole structure, the "cost manager function" would force the *OPTimizer* to address more area to concrete elements, due to its lower cost, producing a non-sense distribution with giant cores to the detriment of steel elements: geometric constraints need to be applied defining a range of area values in which each element can vary in. Especially dealing with CFT sections this option could be a useful tool to select the best ratio concrete/steel in each composite element, knowing at each optimization step its relative influence in respect of the whole behavior and the whole cost of the structure.

Related to this aspect, it would be useful for the *OPTimizer* to be able to read also CFT sections directly from the *Etabs* API, functionality that for now is possible to be gained only defining each composite section by hand in the *OPTimizer* interface.

A remark could be done about the interface with commercial software and in-house software. Although they are not so complex and time-consuming, few steps need to be manually implemented when setting options and area constraints in the *OPTimizer* interface. In addition, after the optimization process the new sections have to be updated manually in a new *Etabs* model. These operations could be streamlined, saving time and improving productivity, developing the *OPTimizer* to make it able to take full advantage of the *Etabs* API (Advanced Programming Interface). Sizing optimization, with all its features and potentials, could indeed be more "integrated" with *Etabs*, or other structural analysis software, so that users could work in a single common interface going quickly back and forth between modelling and sizing optimization.

As conclusion, all the assumptions and considerations carried out so far consider a full division between elastic and plastic analysis: elasticity is the basic hypothesis to perform topology and sizing optimization. However plasticity remains a fundamental topic to be considered in tall building design. Due to an ever increasing number of structural optimization applications to high rise structures in high seismicity areas, the integration with plastic behavior is becoming an important issue to be considered.

As a matter of fact, in the suggested workflow, the plastic and elastic behavior are studied separately in the same structure and the optimization is only addressed to those elements which will remain in the elastic field. Dissipation mechanisms and plastic behavior can be studied only if the involved elements are excluded from the optimization process. The issue of including non-linear materials in the optimization problem has to be explored finding out new methodologies that will allow an elasto-plastic design. The main aspect regards the iterations that the inclusion of this kind of material will required. Once again, therefore, the need of implementing an iterative process is pointed out. New methodologies could be explored to address elasto-plastic optimization from the initial phases of the design process.

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